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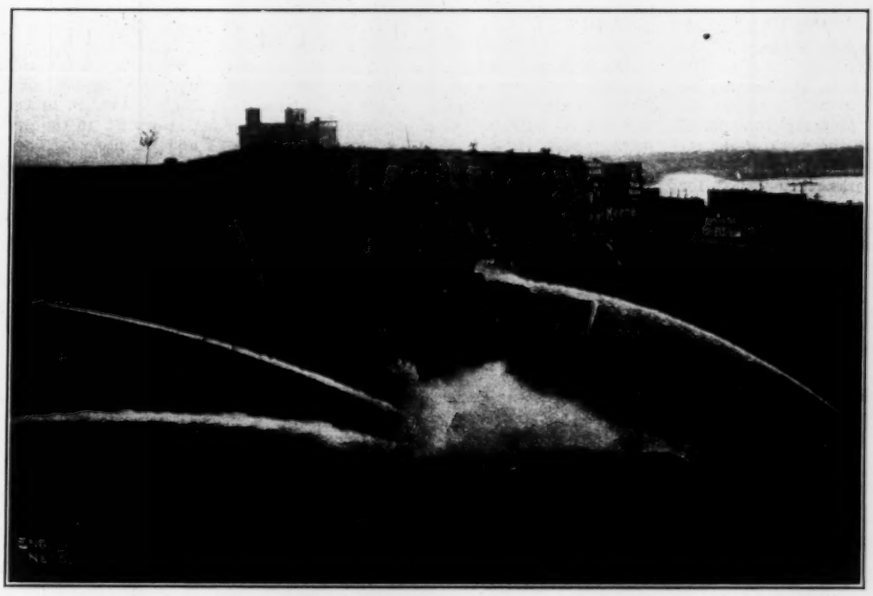
Heavy Regrading by Means of Hydraulic Sluicing at Seattle Wash.

By GEO. HOLMES MOORE,* Secretary Seattle Section, Am. Inst. E. E.

As a result of its rugged topography the city of Seattle, Wash., has a class of engineering problems peculiarly its own. The hills which form its site rise by sharp gradients from the harbor of Elliott Bay to a maximum elevation of approximately 460 ft. at the high-service standpipes of Volunteer Park and Queen Anne Hill. As a consequence of the steep slopes encountered, especially in the business district which clusters about the southeasterly end of the harbor, the problem of regrading has, from the first, been one of vital importance to the city's growth. Another unusual feature in the civic improvement of Seattle, is her almost unprecedented increase in population. From a total of 80,000 people in 1900, this thriving seaport has grown in less than ten years to a population of approximately 320,000.

Because, therefore, of the rapid expansion of the business districts occasioned by this remarkable increase, and also because of the enormous quantities of dirt to be disposed of in a minimum time, the more conventional methods of earth-removal by means of the dirt wagon, the steam shovel and the construction train have long since been superseded by the use of the high-pressure water-jet. It is to be noted, however, that all these methods are still retained as valuable adjuncts to the hydraulic system.



HYDRAULIC GIANTS AT WORK ON DENNY HILL REGRADE, SEATTLE, WASH.
(Looking south into heavy cut at Fourth Ave. and Blanchard St.; top of Hotel Washington in distance.)

Since the inception of regrading in Seattle, the amount of dirt removed is remarkable. More than 14,000,000 cu. yds. of earth and stone have been removed up to the present time, and before the work shall have been completed more than 14,000,000 cu. yds. will have been disposed of.†

The engineering features of the plants installed for the hydraulic reduction of Seattle's hills have received a great deal of attention; and, as an example of the high state of efficiency to which hydraulic methods for earth-removal have been brought, the Denny Hill Regrade, which is now rapidly approaching completion, is well worthy of careful study.

This project comprises an area of 43 city blocks, extending from Pike St. on the south to Denny Way on the north. A very clear idea of the section involved may be obtained by a reference to the plan of the district, Fig. 1, on which the blocks affected are designated by shading.

So urgent was the need for building sites in this section that the first of the regrading, namely, that of the blocks upon which the magnificent new Hotel Washington and Moore Theatre now stand, was done by private enterprise long before the initiation of the project which forms the subject of this article. This first proposition was promoted by a group of capitalists under the leadership of James A. Moore. It involved the destruction of the picturesque Denny Hotel, which for years had been a prominent landmark of the city and which commanded an unmatched outlook upon the harbor and the mountains beyond. A view of the Denny Hotel is shown in the accompanying photograph, which was taken in 1904. (See Fig. 2.) An idea of the original topography of the district is given by the stepped arrangement of the houses shown at the left of the picture. The private cable car line which brought the guests up the sharp incline to the entrance of the hostelry is also shown. A view looking into the same district from approximately the same standpoint, taken five years later, in the fall of 1909, is shown in Fig. 3. The tall building in the background is the Hotel Washington; to the left of this is the Washington Hotel Annex, and to the right is the Moore Theatre, by far the finest theatre in Seattle. The top of the Moore Theatre shown in Fig. 3, is at the approximate elevation of the bottom of the Denny Hotel shown in Fig. 2.

The plant used for the preparation of the site for the Hotel Washington and Moore Theatre was described briefly in the Engineering News of May 16, 1907, under the caption "Hydraulic Excavation in Seattle with a Motor-Driven Pump." Although this pioneer plant was faulty in construction and was the scene of many accidents, it allowed the completion of this first regrade some months ahead of the estimated time, and amply demonstrated the practicability of this method of regrading.

The Denny Hill Regrade.

This first contract having been successfully completed, the more ambitious scheme now known as the Denny Hill Regrade was inaugurated. The legal machinery had been set in motion by the filing of a petition signed by a majority of the property owners in the district and by an ordinance which was approved by the mayor of Seattle on May 23, 1906. The ordinance provided for "the laying off, widening, extending, altering and establishing of Third Ave., between Denny Way and Pike St.; Fourth Ave., between Denny Way and Pike St.; and Fifth Ave., between Westlake Ave. and Denny Way." It further provided for "the changing and establishing of the grades of the following-named streets and avenues, to wit: Third Ave., from Pine St. to Cedar St.; Fourth Ave., from Pine St. to Cedar St.; and Fifth Ave., from Westlake Ave. to Denny Way," together with the severally-named cross streets intersecting them.

Under this proceeding, Third, Fourth and Fifth avenues were widened from 66 ft. to 90 ft., by condemning a 12-ft. strip on each side of the avenues throughout their length, from Pike St. and Westlake Ave. on the south to Denny Way on the north. The awards for the private property so appropriated totaled over $1,500,000.

The grades established by the city engineer's office reduced the gradient on these three thoroughfares to a figure which imposes no obstacle whatever to the traffic which is already beginning to flow along them; as, for example, on Fourth Ave., where, as will be noted from the accompanying profile (Fig. 4), the final maximum grade is 1.2%. This necessitated very heavy

*City Engineer's Office, Seattle, Wash.

[†See Eng. News, Nov. 12, 1908, for detailed illustrated description of the Jackson Hill regrade at Seattle—Ed.]

cuts, running to a maximum, at a point near Fourth Ave. and Blanchard St., of no less than 110 ft. It may be noted that the item of the grading amounted to not far from $1,000,000, bringing the total cost of the improvement up to approximately $2,500,000.

Following the approval of the ordinance as above outlined, suit was brought before the Superior Court to "ascertain the just compensation to be awarded for the taking or damaging of private property" included within the district. The jury impanelled in this case brought in 347 verdicts for amounts aggregating a total of $1,510,958, and on March 13, 1907, a judgment on these verdicts was entered, by which action the city acquired title to the property condemned and was free to proceed with the work of the regrade.

FIG. 1. PLAN OF DENNY HILL REGRADE DISTRICT, SEATTLE, WASH.

The Board of Public Works then called for bids for the grading, according to specifications prepared by the city engineer. The bid of the Rainier Development Co. was accepted as itemized below:

Fixed estimate	$10,000
Clear and grub (lump sum)	1
Earthwork, 3,590,000 cu. yds., at 27 cts. per yd.	969,300
Box drain, 3,500 lin. ft., at 35 cts.	1,225
Total	$980,526

Certain changes were later made in the plans, slightly altering the amount of earth to be removed, and the final estimate of the yardage to be removed for the various purposes is approximately as follows:

	Cu. yds.
Streets	3,314,100
Alleys	174,500
Blocks (private)	1,768,200
Seattle Electric Co.	140,800
Total	5,397,700

With regard to the above itemization, it is to be noted that the amounts given for streets and alleys include also the amounts comprised in the side slopes.

The contract with the Rainier Development Co. on a basis of the bid already given was dated Aug. 17, 1907. It provided the installation of a pumping plant capable of removing 12,000 cu. yds. daily, and required the commencement of the work within a period of 180 days from the date of notice to begin work, and the completion of same within 30 months from that date. The job was later sublet to the firm of Grant Smith & Co. and Stilwell, the assignment of the contract to the latter firm being approved on Aug. 4, 1908, by the Board of Public Works.

Grant Smith & Co. and Stilwell then acquired the original pumping station on the harbor front, commonly known as the Hawley Plant, and at once began the installation of a powerful new plant on Lake Union.

THE HAWLEY PUMPING PLANT.—This comprises a single large pumping unit, consisting of an 800-HP. synchronous motor, belt-connected to a four-stage turbine guaranteed to deliver 3,500 gals. per min. against a pressure of 180 lbs. at the pump. The motor is an early Westinghouse type, which takes 2-phase, 60-cycle current at 2,400 volts. It runs at 330 r. p. m. The pump is a 10-in., four-stage, Worthington turbine, running at 500 r. p. m. The difference in r. p. m. of motor and pump necessitated the use of a belt connection between them. This belt is of leather, 48 ins. wide and 66 ft. long, the shaft centers being approximately 25 ft. apart.

The turbine takes salt water from the Sound, and at extreme high tide the water level is within 2 ft. of the base of the turbine. At extreme low tide, a variation from high tide of not quite 16 ft., the long intake and height of suction-lift tend to cause the turbine to lose its load. Also, the inherent characteristics of the synchronous type required the motor to be brought up to speed under no load, and this in turn necessitated the installation of a heavy clutch coupling between the pump and the motor. Many accidents and shut-downs resulted from trouble with this clutch.

In spite, however, of these and other drawbacks, this original station enabled the completion of the Moore Theatre regrade some months ahead of time. The plant is still in service delivering its 5,000,000 gals. of water per 24-hour day.

THE LAKE UNION PLANT.—The new plant is located on the edge of Lake Union at Eighth Ave. N. and Valley St., at an air-line distance of approximately one mile from the point of maximum cut at Fourth Ave. and Blanchard. It is an excellent example of the most advanced type of the electrically-driven, hydraulic plant as developed for regrade work.

In the one-story frame building which comprises the plant are installed three turbine units, aggregating 1,950 HP. and capable of delivering a total of 12,600 gals. per min. against a pressure head of 180 lbs. This small structure, which is only 20 ft. × 60 ft. in plan, contains also the switchboard panels, solenoid starting-gear and 2,000-volt wiring, as well as the water gages, valves, and all other accessories.

Of the three units, two are the twin-turbine sets which comprised the initial installation for this plant. These are identical in type. Each consists of two four-stage turbines direct-connected to a 650-HP. motor mounted on a common shaft between them. Each turbine is a 10-in. four-stage Worthington, designed to deliver 2,100 gals. per min., against a pressure head of 180 lbs., thus giving 4,200 gals. per min., as the output for each twin.

The third unit is a single, four-stage, 15-in. Jeansville turbine, direct-connected to a 650-HP. motor. This single turbine is designed for the same output as each of the twin sets, or 4,200 gals. per min. against a pressure head of 180 lbs.

All three of the motors are identical, being Westinghouse, type C. C. L., of 650-HP. capacity. They are two-phase induction motors, taking 60-cycle current at 2,000 volts, and running at 690 r. p. m. They are easily started and brought to speed under load, a thing impracticable with the synchronous type. The starting is accomplished by means of a master switch controlling ten solenoids, which, by an auto-transformer scheme of wiring, automatically increase the voltage applied to the motors, one step at a time, and at a predetermined time interval. In this way, the danger of throwing excessive current into the motor during the starting operation is obviated, and the system has proven entirely satisfactory in every way.

FORCE MAINS.—From the turbines, the three discharge pipes, two 10-in. and one 15-in., pass into a common 24-in. wood-stave header or supply pipe which carries the water to the site of the regrade. This supply pipe is the wire-wound, wood-sectional pattern now coming into great demand for many classes of hydraulic work. The same kind of pipe is shown by Fig. 5, except that no lining blocks were used in the force main.

From the plant at Eighth Ave. N. and Valley St., this header or supply-main passes up Eighth Ave. N. to Denny Park and thence along Battery St. to the diversion points at Third Ave. and Fifth Ave. The distance from the plant to these diversion points, measured along the pipe line, is approximately one mile, and the maximum distance from the plant to the nozzles is something less than a mile and a half.

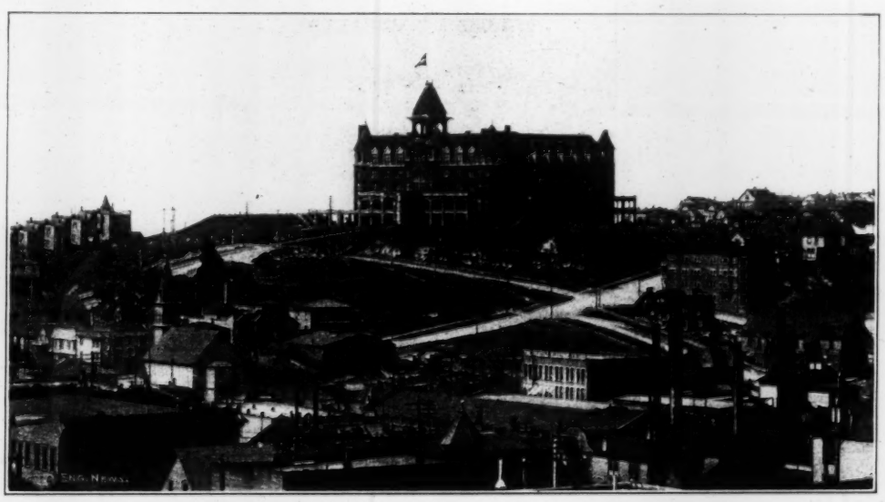


FIG. 2. DENNY HILL, SEATTLE, WASH, IN 1904, BEFORE REGRADING, WITH DENNY HILL HOTEL ON SUMMIT.

At Third and Fifth Aves., the 24-in. main feeds into eleven 12-in. wood-stave laterals which carry the water to the various points of cut. Five of these lead southerly along Third Ave. and six lead southerly along Fifth Ave.

A single 14-in. steel force main carries the water supplied by the Hawley Pumping Plant.

At the diversion points above mentioned, the transition from 24 to 12-in. is accomplished by

FIG 3. DENNY HILL, SEATTLE, WASH., IN 1909, SHOWING WASHINGTON HOTEL AND MOORE ST. THEATER ON NEW LEVEL.

(The old Denny Hotel was about 75 to 80 ft. above the point marked X in front of the large buildings.)

means of simple tee castings, the joints being made by driving the wood pipe into the bells of the casting. The only precaution taken against blowing out at these joints is that the casting of the gate-valve which controls each lateral is strapped to the tee by means of "U"-shaped, round-iron rods. The joints in the straightaway pipe, which follows the contour of the ground, are merely very short lengths of larger sizes of the same kind of pipe, into which the latter are secured by a driving fit.

GIANTS.—The giants are of the type long in use for hydraulic mining, improved, however, so as to allow of being easily swung by the nozzleman through a full circle horizontally and through a very considerable vertical angle as well. This is accomplished by means of a self-sealing ball-and-socket design of great ruggedness and simplicity. Detachable screw-coupling nozzles are used, with diameters varying from 3 to 3½ ins., to produce a jet of greater or less velocity as desired.

DISCHARGE OF SLUICED MATERIAL.—At the beginning of the regrade it was necessary to provide a means of discharge into the harbor. In order to avoid disturbance of the traffic which flows along First and Second avenues, a tunnel was driven under them. This tunnel runs along Bell St. from the upper end between Second and Third avenues, indicated on plan (Fig. 1) as "Open Cut," to a point just east of Elliott Ave., indicated on the plan as "Mouth." This tunnel was driven by the hydraulic method, the water used being taken from the city mains. It was timbered and lined with rough lumber and has a present effective section approximately 3½ ft. wide by 6 ft. high. From the mouth of this tunnel the mixture of mud and water is carried to the points of discharge through a timber flume, 3 ft. wide inside by 4 ft. high. This flume passes above the tracks and roadway of Railroad Ave. by means of high trestles and a short-span timber-truss bridge. From Railroad Ave. to the points of discharge in deep water, the flume is carried on timber bents supported on piling.

It was necessary to secure permission from the U. S. Government for the discharge into the harbor, and as this permission was conditional upon the maintenance of a minimum depth of water of 45 ft. at low tide, the point of discharge has shifted steadily outward toward the deep waters of the bay. In order to allow the pile drivers to operate without interrupting the progress of the regrade, provision has been made for three sluiceways, any one or two of which may be in use while the driver is extending the third.

Besides the hydraulic jets, which are steadily eating into the breasts of the various cuts, a construction railway has been constantly in operation, hauling dirt from the steam-shovels and dumping it into the open cut previously mentioned. Two trains, consisting of four side-dump cars each, are operated on this single-track road, with a third locomotive to help out on the steeper grades.

In order to break up the dirt from these trains and wash it into the tunnel, two small-size giants have been installed at this point. These are supplied by a 2-in. and a 4-in. iron pipe, respectively.

One feature of the work which is favorable to its early completion is the small percentage of rock occurring in the mass to be removed. And although the greater part of the mass is a very hard, blue-black clay—so hard and closely-compacted, in fact, that in the earlier attempts it was seriously doubted that it could be handled hydraulically at all—still the hardest part of the work is now past. And at this time, the close of 1909, the entire project is reported to be more than three-fourths completed.

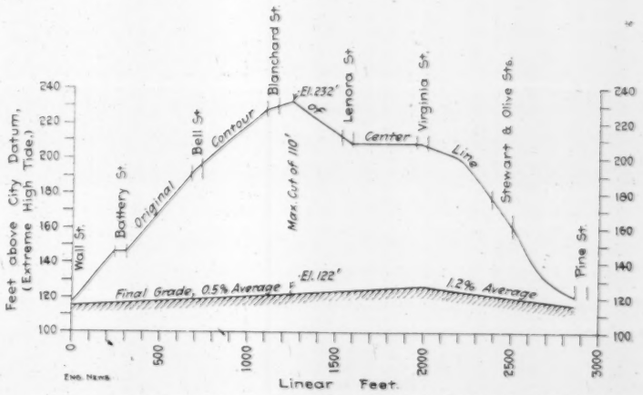


FIG. 4. PROFILE OF FOURTH AVE., SEATTLE, WASH., SHOWING LEVELS BEFORE AND AFTER DENNY HILL REGRADE.

SLUICEWAY LININGS.—An interesting point in connection with all hydraulic regrading, and one hardly to be considered important except in the light of experience, is the extreme diffi-

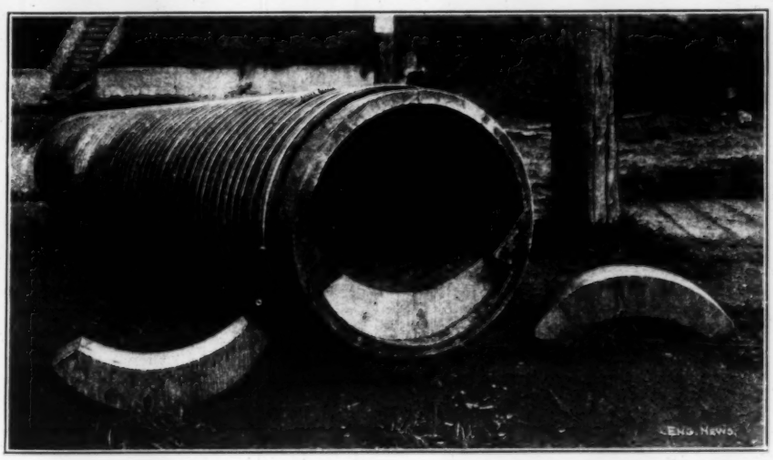


Fig. 5. Illustrating Blocks and Retaining Ribs Before Use.

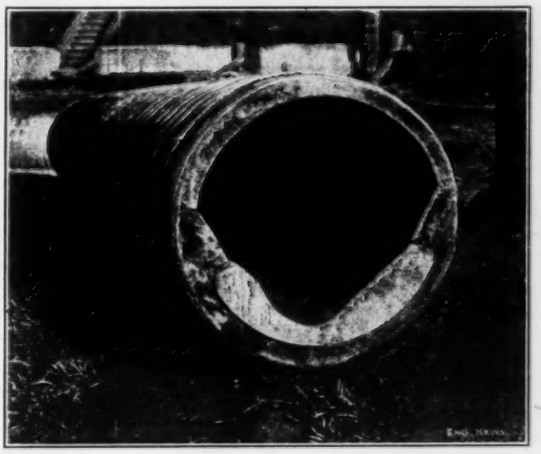


Fig. 6. Showing Effect of Use and Wear on Blocks.

FIGS. 5 AND 6. WIRE-WOUND WOOD-STAVE PIPE, WITH SPECIAL INVERT LINING OF WOOD BLOCKS SET WITH GRAIN VERTICAL, USED ON SEATTLE REGRADE WORK.

culty encountered of finding any substance which will resist the high attrition in the sluiceways. This point was well brought out in a paper presented before the Pacific Northwest Society of Enginers, on the difficulties so encountered in the construction of the mammoth embankment on the Chicago, Milwaukee & Puget Sound Ry. near North Bend, Wash., a brief outline of which project appeared in Engineering News of March 12, 1908. In this case, sheet-iron, cast-iron and even steel plate was tried in the sluiceways in order to take up the wear of the rolling sand and boulders. All of these wore through with surprising rapidity. The remedy finally hit upon was removable wood blocks, placed end up to the wear. Where these are used in wood-stave pipe, two of the side-sections of same are cut larger to act as retaining ribs (see Figs. 5 and 6) and the blocks are turned in a lathe so as to give a maximum length of block at the bottom of the pipe, or point of greatest wear. This type of lining, together with the effect of wear upon the wood blocks, is clearly shown in the Figs. 5 and 6. So important is this point, that the use of wood-block lining has been patented by a local firm, and one successful damage-suit has already been brought against infringers.

Where the wood-block lining is used in flumes, tunnels, etc., oblong blocks, 6 ins. long, are cut from 6 × 12-in. rough lumber, and the bottom of the sluiceway is lined with these, laid end up to wear. Evidently, when these wear out, it is a simple matter to remove them and replace with new blocks.

In order to hasten the sluicing, both by the loosening of large masses and by the breaking up of the more closely-compacted lumps of the shale-like clay, blasting powder is used throughout the area under regrade. Comparatively small charges are used, however, and very little disturbance has so far resulted from this cause.

These, then, are the salient features of a regrading project which is remarkable even for Seattle, a project involving the removal of nearly 5,400,000 cu. yds. of material, an amount surpassing by a million cubic yards that of its nearest predecessor, the Jackson St. regrade; and it is very probable that, both in point of the amount of material handled and in respect to the maximum depth of cut attained, the Denny Hill Regrade above described is the largest municipal regrading problem ever carried to a successful completion.

The plans for this improvement were drawn up under the direction of Mr. R. H. Thomson, City Engineer for the city of Seattle, and the work has been carried forward under the immediate supervision of Mr. D. W. McMorris, Principal Assistant City Engineer, to both of whom as well as to Mr. O. A. Piper, District City Engineer, the writer acknowledges his obligation for the data necessary for the preparation of this paper.

The contracting company having the work in charge is the local firm of contractors and engineers operating under the firm name of Grant Smith & Co. and Stilwell.

Some Mooted Points in Reinforced Concrete Design.*

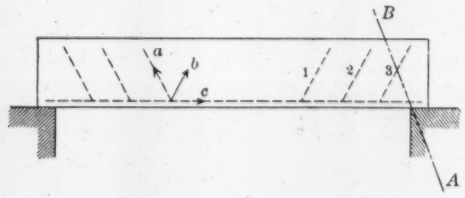
By EDWARD GODFREY,† M. Am. Soc. C. E.

Reinforced concrete engineering has fairly leaped into prominence and apparently into full growth, but it still wears some of its swaddling-bands. Some of the garments which it borrowed from sister forms of construction in its short infancy still cling to it, and, while these were, perhaps, the best makeshifts under the circumstances, they fit badly and should be discarded. It is some of these misfits and absurdities that the writer would like to bring before the engineering profession.

The first point to which attention is called is illustrated in the figure herewith. It concerns sharp bends in reinforcing rods in concrete. The figure shows a reinforced concrete design, one held out, in nearly all books on the subject, as model. The reinforcing rod is bent up at a sharp angle, and then may or may not be bent again and run parallel with the top of the beam. At the bend is a condition which resembles that of a hog-chain or truss-rod around a queen-post. The reinforcing rod is the hog-chain or the truss-rod. Where is the queen-post? Suppose this rod has a section of 1 sq. in. and an inclination of 60° with the horizontal, and that its unit stress is 16,000 lbs. per sq. in. The forces, *a* and *b*, are then 16,000 lbs. The force, *c*, must be also 16,000 lbs. What is to take this force, *c*, of 16,000 lbs.? There is nothing but concrete. At 500 lbs. per sq. in., this force would require an area of 32 sq. ins. Will some advocate of this type of design please state where this area can be found? It must, of necessity, be in contact with the rod, and, for structural reasons, because of the lack of stiffness in the rod, it would have to be close to the point of bend. If analogy to the queen-post fails so completely, because of the almost complete absence of the post, why should not this borrowed garment be discarded?

If this same rod be given a gentle curve of a radius twenty or thirty times the diameter of the rod, the side unit pressure will be from one-twentieth to one-thirtieth of the unit stress on the steel. This being the case, and being a simple principle of mechanics which ought to be thoroughly understood, it is astounding that engineers should perpetrate the gross error of making a sharp bend in a reinforcing rod under stress.

The second point to which attention is called may also be illustrated by the figure. The rod marked 3 is also like the truss-rod of a queen-post truss in appearance, because it ends over the support and has the same shape. But the analogy ends with appearance, for the function of a truss-rod in a queen-post truss is not performed by such a reinforcing rod in concrete, for other reasons than the absence of a post. The truss-rod receives its stress by a suitable connection at the end of the rod and over the support of the beam. The reinforcing rod, in this standard beam, ends abruptly at the very point where it is due to receive an important element of strength, an element which would add enormously to the strength and safety of many a beam, if it could be introduced.



Of course a reinforcing rod in a concrete beam receives its stress by increments imparted by the grip of the concrete; but these increments can only be imparted where the tendency of the concrete is to stretch. This tendency is greatest near the bottom of the beam, and when the rod is bent up to the top of the beam, it is taken out of the region where the concrete has the greatest tendency to stretch. The function of this rod as reinforcement of the bottom flange of the beam, is interfered with by bending it up in this manner, as the beam is left without bottom-flange reinforcement, as far as that rod is concerned, from the point of bend to the support.

It is true that there is a shear or a diagonal tension in the beam, and the diagonal portion of the rod is apparently in a position to take this tension. This is just such a force as the truss-rod in a queen-post truss must take. Is this reinforcing rod equipped to perform this office? The beam is apt to fail in the line, *A B*. In fact, it is apt to crack from shrinkage on this or almost any other line, and to leave the strength dependent on the reinforcing steel. Suppose such a crack should occur. The entire strength of the beam would be dependent on the grip of the short end of Rod 3 to the right of the line *A B*. The grip of this short piece of rod is so small and precarious, considering the important duty it has to perform, that it is astounding that designers, having any care for the permanence of their structures, should consider for an instant such features of design, much less incorporate them in a building in which life and property depend on them.

The third point to which attention is called, is the feature of design just mentioned in connection with the bent-up rod. It concerns the anchorage of rods by the embedment of a few inches of their length in concrete. This most flagrant violation of common sense has its most conspicuous example in large engineering works, where of all places better judgment should prevail. Many retaining walls have been built which have, as an essential feature, a great number of rods which cannot possibly develop their strength, and might as well be of much smaller dimensions. These rods are the vertical and horizontal rods in the counterfort of the retaining wall. The retaining wall generally consists of a front curtain wall and a horizontal slab joined at intervals by ribs or counterforts. The manifest and only function of the rib or counterfort is to tie together the curtain wall and the horizontal slab. That it is or should be of concrete is because the steel rods which it contains need protection. It is clear that, apart from the cracking of the concrete of the rib, the only thing which would produce rupture in the wall is the pulling out of the short ends of the reinforcing rods in the counterfort. It can be stated just as positively that a set of rods running from the front wall to the horizontal slab, and anchored into each in such a manner as would be adopted were these slabs suspended on the rods, is the only rational and the only efficient design possible.

The fourth point concerns shear in steel rods embedded in concrete. For decades, specifications for steel bridges have gravely given a unit shear to be allowed on bridge pins, and every bridge engineer knows or ought to know that, if a bridge pin is properly proportioned for bending and bearing, there is no possibility of its being weak from shear. The centers of bearings cannot be brought close enough together to reduce the size of the pin to where its shear need be considered, because of the width required for bearing on the parts. Concrete is about one-thirtieth as strong as steel in bearing. There is, therefore, somewhat less than one-thirtieth of a reason for specifying any shear on steel rods embedded in concrete.

The gravity of the situation is not so much the serious manner in which this unit of shear in steel is written in specifications and building codes for reinforced concrete work (it does not mean anything in specifications for steelwork, because it is ignored), but it is apparent when designers soberly use these absurd units, and proportion shear rods accordingly.

Many designers actually proportion shear rods for shear, shear in the steel at units of 10,000 or 12,000 lbs. per sq. in.; and the blame for this dangerous practice can be laid directly to the literature on reinforced concrete. Shear rods are given as standard features in the design of reinforced concrete beams. In the Joint Report of the Committee of the various engineering societies, a method for proportioning shear members is given. The stress, or shear per shear member, is the longitudinal shear which would occur in the space from member to member. No hint is given as to whether these bars are in shear or tension; in fact, either would be absurd and impossible without greatly overstressing some other part. This is just a sample of the state of the literature on this important subject. Shear bars will be taken up more fully in subsequent paragraphs.

The fifth point concerns vertical stirrups in a beam. These stirrups are conspicuous features in the designs of reinforcing concrete beams. Explanations of how they act are conspicuous in the literature on reinforced concrete by its total absence. By stirrups are meant the so-called shear rods strung along a reinforcing rod. They are usually U-shaped and looped around the rod.

It is a common practice to count these stirrups in the shear, taking the horizontal shear in a beam. In a plate girder, the rivets connecting the flange to the web take the horizontal shear or the increment to the flange stress. Compare two ¾-in. rivets tightly driven into holes in a steel angle, with a loose vertical rod, ¾-in. in diameter, looped around a reinforcing rod in a concrete beam, and a correct comparison of methods of design in steel and reinforced concrete, as they are commonly practiced, is obtained.

These stirrups can take but little hold on the reinforcing rods—and this must be through the medium of the concrete—and they can take but little shear. Some writers, however, hold the opinion that the stirrups are in tension and not in shear, and some are bold enough to compare them with the vertical tension members of a Howe truss. Imagine a Howe truss with the vertical tension members looped around the bottom chord and run up to the top chord without any connection, or hooked over the top chord; then compare such a truss with one in which the end of the rod is upset and receives a nut and large washer bearing solidly against the chord. This gives a comparison of methods of design in wood and reinforced concrete, as they are commonly practiced.

Anchorage or grip in the concrete is all that can be counted on, in any event, to take up the tension of these stirrups, but it requires an embedment of from 30 to 50 diameters of a rod to develop its full strength. Take 30 to 50 diameters from the floating end of these shear members, and, in some cases, nothing or less than nothing will be left. In any case the point at which the shear member, or stirrup, is good for its full value, is far short of the centroid of compression of the beam, where it should be; in most cases it will be nearer the bottom of the beam. In a Howe truss, the vertical tension members having their end connections near the bottom chord, would be equivalent to these shear members.

The sixth point concerns the division of stress into shear members. Briefly stated, the common method is to assume each shear member as taking the horizontal shear occurring in the space from member to member. As already stated, this is absurd. If stirrups could take shear, this method would give the shear per stirrup, but even advocates of this method acknowledge that they cannot. To apply the common analogy of a truss: each shear member would represent a tension web member

*Portion of a paper read before the American Society of Civil Engineers, March 16, 1910. Reprinted from the Proceedings of the Society, v. 36, No. 2, February, 1910, p. 190.

†Monongahela Bank Bldg., Pittsburg, Pa.

in the truss, and each would have to take all the shear occurring in a section through it.

If, for example, shear members were spaced half the depth of a beam apart, each would take half the shear by the common method. If shear members take vertical shear, or if they take tension, what is between the two members to take the other half of the shear? There is nothing in the beam but concrete and the tension rod between the two shear members. If the concrete can take the shear, why use steel members? It is not conceivable that an engineer should seriously consider a tension rod in a reinforced-concrete beam as carrying the shear from stirrup to stirrup.

The logical deduction from the proposition that shear rods take tension is that the tension rods must take shear, and that they must take the full shear of the beam, and not only a part of it. For these shear rods are looped around or attached to the tension rods, and since tension in the shear rods would logically be imparted through the medium of this attachment, there is no escaping the conclusion that a large vertical force (the shear of the beam) must pass through the tension rod. If the shear member really relieves the concrete of the shear, it must take it all. If, as would be allowable, the shear rods take but a part of the shear, leaving the concrete to take the remainder, that carried by the rods should not be divided again, as is recommended by the common method.

Bulletin No. 29 of the University of Illinois Experiment Station shows by numerous experiments, and reiterates again and again, that shear rods do not act until the beam has cracked and partly failed. This being the case, a shear rod is an illogical element of design. Any element of a structure, which cannot act until failure has started, is not a proper element of design. In a steel structure a bent plate which would straighten out under a small stress and then resist final rupture, would be a menace to the rigidity and stability of the structure. This is exactly analogous to shear rods which cannot act until failure has begun.

When the man who tears down by criticism fails to point out the way to build up, he is a destructive critic. If, under the circumstances, designing with shear rods had the virtue of being the best thing to do with the steel and concrete disposed in a beam, as far as experience and logic in their present state could decide, nothing would be gained by simply criticizing this method of design. But logic and tests have shown a far simpler, more effective, and more economical means of disposing of the steel in a reinforced-concrete beam.

In shallow beams there is little need of provision for taking shear by any other means than the concrete itself. The writer has seen a reinforced slab support a very heavy load by simple friction, for the slab was cracked close to the supports. In slabs, shear is seldom provided for in the steel reinforcement. It is only when beams begin to have a depth approximating one-tenth of the span that the shear in the concrete becomes excessive and provision is necessary in the steel reinforcement. Years ago, the writer recommended that, in such beams, some of the rods be curved up toward the ends of the span and anchored over the support. Such reinforcement completely relieves the concrete of all shearing stress, for the stress in the rod will have a vertical component equal to the shear. The concrete will rest in the rod as a saddle, and the rod will be like the cable of a suspension span. The concrete could be in separate blocks with vertical joints, and still the load would be carried safely.

By end anchorage is not meant an inch or two of embedment in concrete, for an iron vise would not hold a rod for its full value by such means. Neither does it mean a hook on the end of the rod. A threaded end with a bearing washer, and a nut and a lock-nut to hold the washer in place, is about the only effective means, and it is simple and cheap. Nothing is as good for this purpose as plain round rods, for no other shape affords the same simple and effective means of end connection. In a line of beams, end to end, the rods may be extended into the next beam, and there act to take the top-flange tension, while at the same time finding anchorage for the principal beam stress.

The simplicity of this design is shown still further by the absence of a large number of little pieces in a beam box, as these must be held in their proper places, and as they interfere with the pouring of the concrete.

Bulletin 29 of the University of Illinois Experiment Station contains a record of tests of reinforced concrete beams of this sort. They failed by the crushing of the concrete or by failure in the steel rods, and nearly all the cracks were in the middle third of the beams, whereas beams rich in shear rods cracked principally in the end thirds, that is, in the neighborhood of the shear rods. The former failures are ideal, and are easier to provide against. A crack in a beam near the middle of the span is of little consequence, whereas one near the support is a menace to safety.

The seventh point of common practice to which attention is called, is the manner in which bending moments in so-called continuous beams are juggled to reduce them to what the designer would like to have them. This has come to be almost a matter of taste, and is done with as much precision or reason as geologists guess at the age of a fossil in millions of years.

If a line of continuous beams be loaded uniformly, the maximum moments are negative and are over the supports. Who ever heard of a line of beams in which the reinforcement over the supports was double that at mid-spans? The end support of such a line of beams cannot be said to be fixed, but is simply supported, hence the end beam would have a negative bending moment over next to the last support equal to that of a simple span. Who ever heard of a beam being reinforced for this? The common practice is to make a reduction in the bending moment, at the middle of the span, to about that of a line of continuous beams, regardless of the fact that they may not be continuous or even contiguous, and in spite of the fact that the loading of only one gives quite different results, and may give results approaching those of a simple beam.

If the beams be designed as simple beams—taking the clear distance between supports as the span and not the centers of bearings or the centers of support—and if a reasonable top reinforcement be used over these supports to prevent cracks, every requirement of good engineering is met. Under extreme conditions such construction might be heavily stressed in the steel over the supports. It might even be overstressed in this steel, but what could happen? Not failure, for the beams are capable of carrying their load individually, and even if the rods over the supports were severed—a thing impossible because they cannot stretch out sufficiently—the beams would stand.

Continuous beam calculations have no place whatever in designing stringers of a steel bridge, though the end connections will often take a very large moment, and, if calculated as continuous, will be found to be strained to a very much larger moment. Who ever heard of a failure because of continuous beam action in the stringers of a bridge? Why cannot reinforced concrete engineering be placed on the same sound footing as structural steel engineering?

The eighth point concerns the spacing of rods in a reinforced-concrete beam. It is common to see rods bunched in the bottom of such a beam with no regard whatever for the ability of the concrete to grip the steel, or to carry the horizontal shear incident to their stress, to the upper part of the beam. It is reasoning of the most elementary kind, which shows that if shear and adhesion are equal, the width of a reinforced concrete beam should be equal to the sum of the peripheries of all reinforcing rods gripped by the concrete. The width of the beam is the measure of the shearing area above the rods, taking the horizontal shear to the top of the beam, and the peripheries of the rods are the measure of the gripping or adhesion area.

Analysis which examines a beam to determine whether or not there is sufficient concrete to grip the steel and to carry the shear, is about at the vanishing point in nearly all books on the subject. Such misleading analysis as that just cited is worse than nothing.

The ninth point concerns the T-beam. Excessively elaborate formulas are worked out for the T-beam, and haphazard guesses are made as to how much of the floor slab may be considered in the compression flange. If a fraction of this mental energy were directed toward a logical analysis of the shear and gripping value of the stem of the T-beam, it would be found that, when the stem is given its proper width, little, if any, of the floor slab will have to be counted in the compression flange, for the width of concrete which will grip the rods properly will take the compression incident to their stress.

The tenth point concerns elaborate theories and formulas for beams and slabs. Formulas are commonly given with 25 or 30 constants and variables to be estimated and guessed at, and are based on assumptions which are inaccurate and untrue. One of these assumptions is that the concrete is initially unstressed. This is quite out of reason, for the shrinkage of the concrete on hardening puts stress in both concrete and steel. One of the coefficients of the formulas is that of the elasticity of the concrete. No more variable property of concrete is known than its coefficient of elasticity, which may vary from 1,000,000 to 5,000,000 or 6,000,000; it varies with the intensity of stress, with the kind of aggregate used, with the amount of water used in mixing, and with the atmospheric condition during setting. The unknown coefficient of elasticity of concrete and the non-existent condition of no initial stress, vitiate entirely formulas supported by these two props.

Here again destructive criticism would be vicious if these mathematical gymnasts were giving the best or only solution which present knowledge could produce, or if the critic did not point out a substitute. The substitute is so simple of application, in such agreement with experiments, and so logical in its derivation, that it is surprising that it has not been generally adopted. The neutral axis of reinforced-concrete beams under safe loads is near the middle of the depth of the beams. If, in all cases, it be taken at the middle of the depth of the concrete beam, and if variation of intensity of stress in the concrete be taken as uniform from this neutral axis up, the formula for the resisting moment of a reinforced-concrete beam becomes extremely simple and no more complex than that for a rectangular wooden beam.

The eleventh point concerns complex formulas for chimneys. It is a simple matter to find the tensile stress in that part of a plain concrete chimney between two radii on the windward side. If in this space there is inserted a rod which is capable of taking that tension at a proper unit, the safety of the chimney is assured, as far as that tensile stress is concerned. Why should frightfully complex formulas be proposed, which bring in the unknowable modulus of elasticity of concrete and can only be solved by stages or dependence on the calculations of some one else?

The twelfth point concerns deflection calculations. As is well known, deflection does not play much of a part in the design of beams. Sometimes, however, the passing requirement of a certain floor construction is the amount of deflection under a given load. Professor Gaetano Lanza has given some data on recorded deflections of reinforced-concrete beams.* He has also worked out the theoretical deflections on various assumptions. An attempt to reconcile the observed deflections with one of several methods of calculating stresses led him to the conclusion that:

> The observations made thus far are not sufficient to furnish the means for determining the actual distribution of the stresses, and hence for the deduction of reliable formulae for the computation of the direct stresses, shearing stresses, diagonal stresses, deflections, position of the neutral axis, etc., under a given load.

Professor Lanza might have gone further and said that the observations made thus far are sufficient to show the hopelessness of deriving a formula that will predict accurately the deflection of a reinforced-concrete beam. The wide variation shown by two beam tests cited by him, in which the beams were identical, is, in itself, proof of this.

Taking the data of these tests, and working out the modulus of elasticity from the recorded deflections, as though the beams were of plain concrete, values are found for this modulus which are not out of agreement with the value of that variable modulus as determined by other means. Therefore, if the beams be considered as plain concrete beams, and an average value be assumed for the modulus or coefficient of elasticity, a deflection may be found by a simple calculation which is an average of that which may be expected. Here again, simple theory is better than complex, because of the ease with which it may be applied, and because it gives results which are just as reliable.

The thirteenth point concerns the elastic theory as applied to a reinforced-concrete arch. This theory treats a reinforced-concrete arch as a spring. In order to justify its use, the arch or spring is considered as having fixed ends. The results obtained by the intricate methods of the elastic theory and the simple method of the equilibrium polygon, are too nearly identical to justify the former when the arch is taken as hinged at the ends.

The assumption of fixed ends in an arch is a most extravagant one, because it means that the abutments must be rigid, that is, capable of taking bending moments. Rigidity in an abutment is only effected by a large increase in bulk, whereas strength in an arch ring is greatly augmented by the addition of a few inches to its thickness. By the elastic theory, the arch ring does not appear to need as much strength as by the other method, but additional stability is needed in the abutments in order to take the bending moments. This latter feature is not dwelt on by the elastic theorists.

In the ordinary arch, the criterion by which the size of abutment is gaged, is the location of the line of pressure. It is difficult and expensive to obtain depth enough in the base of the abutment to keep this line within the middle third, when only the thrust of the arch is considered. If, in addition to the thrust, there is a bending moment which, for many conditions of loading, further displaces the line of pressure toward the critical edge, the difficulty and expense are increased. It cannot be gainsaid that a few cu. yds. of concrete added to the ring of an arch will go much further toward strengthening the arch than the same amount of concrete added to the two abutments.

Apart from the objection that the elastic theory, instead of showing economy by cutting down the thickness of the arch ring, would show the very opposite if fully carried out, there are objections of greater weight, objections which strike at the very foundation of the theory as applied to reinforced concrete. In the elastic theory, as in the intricate beam theory commonly used, there is the assumption of an initial unstressed condition of the materials. This is not true of a beam and is still further from the truth in the case of an arch. Besides shrinkage of the concrete, which always produces unknown initial stresses, there is a still more potent cause of initial stress, namely, the settlement of the arch when the forms are removed. If the initial stresses are unknown, ideal determinations of stresses can have little meaning.

The elastic theory stands or falls according as one is

*"Stresses in Reinforced-Concrete Beams," Journal, Am. Soc. Mech. Engrs., Mid-October, 1909.

able or unable to calculate accurately the deflection of a reinforced-concrete beam; and it is an impossibility to calculate this deflection even approximately. The tests cited by Professor Lanza show the utter disagreement in the matter of deflections. Of those tested, two beams which were identical showed results almost 100% apart. A theory grounded on such a shifting foundation does not deserve serious consideration. Professor Lanza's conclusions, quoted under the twelfth point, have special meaning and force when applied to a reinforced-concrete arch; the actual distribution of the stresses cannot possibly be determined, and complex cloaks of arithmetic cannot cover this fact. The elastic theory, far from being a reliable formula, is false and misleading in the extreme.

The fourteenth point refers to temperature calculations in a reinforced-concrete arch. These calculations have no meaning whatever. To give the grounds for this assertion would be to reiterate much of what has been said under the subject of the elastic arch. If the unstressed shape of an arch cannot be determined because of the unknown effect of shrinkage and settlement, it is a waste of time to work out a slightly different unstressed shape due to temperature variation, and it is a further waste of time to work out the supposed stresses resulting from deflecting that arch back to its actual shape.

If no other method of finding the approximate stresses in an arch existed, the elastic theory might be classed as the best available; but this is not the case. There is a method which is both simple and reliable. Accuracy is not claimed for it, and hence it is in accord with the more or less uncertain materials dealt with. Complete safety, however, is assured, for it treats the arch as a series of blocks, and the cementing of these blocks into one mass cannot weaken the arch. Reinforcement can be proportioned in the same manner as for chimneys, by finding the tension exerted to pull these blocks apart and then providing steel to take that tension.

The fifteenth point concerns steel in compression in reinforced-concrete columns or beams. It is common practice—and it is recommended in the most pretentious works on the subject—to include in the strength of a concrete column slender longitudinal rods embedded in the concrete. To quote from one of these works:

> The compressive resistance of a hooped member exceeds the sum of the following three elements: (1) The compressive resistance of the concrete without reinforcement. (2) The compressive resistance of the longitudinal rods stressed to their elastic limit. (3) The compressive resistance which would have been produced by the imaginary longitudinals at the elastic limit of the hooping metal, the volume of the imaginary longitudinals being taken as 2.4 times that of the hooping metal.

This does not stand the test, either of theory or practice; in fact, it is far from being true. Its departure from the truth is great enough and of serious enough moment to explain some of the worst accidents in the history of reinforced concrete.

It is a nice theoretical conception that the steel and the concrete act together to take the compression, and that each is accommodating enough to take just as much of the load as will stress it to just the right unit. Here again, initial stress plays an important part. The shrinkage of the concrete tends to put the rods in compression, the load adds more compression on the slender rods and they buckle, because of the lack of any adequate stiffening, long before the theorists' ultimate load is reached.

There is no theoretical or practical consideration which would bring in the strength of the hoops after the strength of the concrete between them has been counted. All the compression of a column must, of necessity, go through the disk of concrete between the two hoops (and the longitudinal steel). No additional strength in the hoops can affect the strength of this disk, with a given spacing of the hoops. It is true that shorter disks will have more strength, but this is a matter of the spacing of the hoops and not of their sectional area, as the above quotation would make it appear.

Besides being false theoretically, this method of investigating phantom columns with real strength is wofully lacking in practical foundation. Even the assumption of reinforcing value to the longitudinal steel rods is not at all borne out in tests. Designers add enormously to the calculated strength of concrete columns when they insert some longitudinal rods. It appears to be the rule that real columns are weakened by the very means which these designers invest with reinforcing properties. whether or not it is the rule, the mere fact that many tests have shown these so-called reinforced-concrete columns to be weaker than similar plain concrete columns, is amply sufficient to condemn the practice of assuming strength which may not exist. Of all parts of a building, the columns are the most vital. The failure of one column will, in all probability, carry with it many others stronger than itself, whereas a weak and failing slab or beam does not put an extra load and shock on the neighboring parts of a structure.

In Bulletin No. 10 of the University of Illinois Experiment Station,* a plain concrete column, 9 × 9 in. × 12 ft., stood an ultimate crushing load of 2,004 lbs. per sq. in. Column 2, identical in size, and having four ⅝-in. rods embedded in the concrete, stood 1,557 lbs. per sq. in. So much for longitudinal rods without hoops. This is not an isolated case, but appears to be the rule; and yet, in reading the literature on the subject, one would be led to believe that longitudinal steel rods in a plain concrete column add greatly to the strength of the column.

A paper, by Mr. M. O. Withey, before the American Society for Testing Materials, in 1909, gave the results of some tests on concrete-steel and plain concrete columns. (The term, concrete-steel, is used because this particular combination is not "reinforced" concrete.) One group of columns, namely, *W*1 to *W*3, 10½ ins. in diameter, 102 ins. long, and circular in shape, stood an average ultimate load of 2,600 lbs. per sq. in. These columns were of plain concrete. Another group, namely, *E*1 to *E*3, were octagonal in shape, with a short diameter (12 ins.), their length being 120 ins. These columns contained nine longitudinal rods, ⅝-in. in diameter, and ¼-in. steel rings every foot. They stood an ultimate load averaging 2,438 lbs. per sq. in. This is less than the column with no steel and with practically the same ratio of slenderness.

In some tests on columns made by the Department of Buildings, of Minneapolis, Minn.,* Test *A* was a 9 × 9-in. column, 9 ft. 6 ins. long, with ten longitudinal, round rods, ½-in. in diameter, and 1½ × 3/16-in. circular bands (having two ½-in. rivets in the splice), spaced 4 ins. apart, the circles being 7 ins. in diameter. It carried an ultimate load of 130,000 lbs., which is much less than half "the compressive resistance of a hooped member," worked out according to the authoritative quotation before given. Another similar column stood a little more than half that "compressive resistance." Five of the seventeen tests on the concrete-steel columns, made at Minneapolis, stood less than the plain concrete columns. So much for the longitudinal rods, and for hoops which are not close enough to stiffen the rods; and yet, in reading the literature on the subject, any one would be led to believe that longitudinal rods and hoops add enormously to the strength of a concrete column.

The sixteenth indictment against common practice is in reference to flat slabs supported on four sides. Grashof's formula for flat plates has no application to reinforced-concrete slabs, because it is derived for a material strong in all directions and equally stressed. The strength of concrete in tension is almost nil, at least, it should be so considered. Poisson's ratio, so prominent in Grashof's formula, has no meaning whatever in steel reinforcement for a slab, because each rod must take tension only; and instead of a material equally stressed in all directions, there are generally sets of independent rods in only two directions. In a solution of the problem given by a high English authority, the slab is assumed to have a bending moment of equal intensity along its diagonal. It is quite absurd to assume an intensity of bending clear into the corner of a slab, and on the very support equal to that at its center. A method published by the writer some years ago has not been challenged. By this method strips are taken across the slab and the moment in them is found, considering the limitations of the several strips in deflection imposed by those running at right angles therewith. This method shows (as tests demonstrate) that when the slab is oblong, reinforcement in the long direction rapidly diminishes in usefulness. When the ratio is 1:1½, reinforcement in the long direction is needless, since that in the short direction is required to take its full amount. In this way French and other regulations give false results, and fail to work out.

If the writer is wrong in any or all of the foregoing points, it should be easy to disprove his assertions. It would be better to do this than to ridicule or ignore them, and it would even be better than to issue reports signed by authorities, which commend the practices herein condemned.

*Page 14, column 8.

*Engineering News, Dec. 3, 1908.

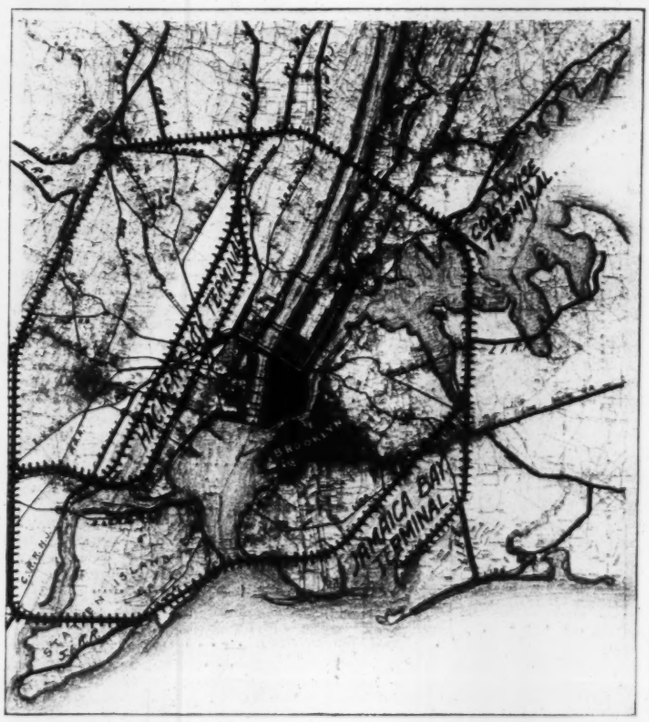
The Cost of Terminal Freight Handling in the Port of New York.

[Contributed.]

The editorial in Engineering News of March 3 on "The Cost of Terminal Handling in Freight Transportation" is most timely in directing attention to the lack of knowledge that prevails among many railroad officers on the relation of terminal expenses to road costs.

It is of course well known that the best brains in the calling and millions of money have been devoted to grade reductions, straightening alinement, modernizing classification yards, increasing the capacity and efficiency of motive power and cars, and in many other ways effecting improvements and reforms that have resulted in making the cost of main line railway haul in this country the lowest in the world.

On the other hand, our terminal methods are perhaps the most archaic, especially in great cities, where the cost of collecting, distributing and dispatching freight between the end of the road haul and the shipper or consumer is costly almost beyond belief, largely due to a division of responsibility between shipper and carrier, and to an absence of the intelligent thought and energy that in other phases of the railroad problem have so successfully advanced the art of transportation.



PROPOSED OUTER BELT LINE RAILWAY SYSTEM FOR NEW YORK CITY AND THE PORT OF NEW YORK.

Not only, as Engineering News has forcefully demonstrated, are terminal costs excessive, but the time consumed by freight shipments between a city like New York and tributary regions within a radius of three or four hundred miles is astonishingly slow—frequently a week or two—as compared with the prompt movement of similar classes of freight in European countries.

To cure these faults there appear to be a variety of possible methods, as by the installation of labor-saving machinery, by the use of outlying belt railways, and by improved methods of local distribution.

Perhaps the most fertile field for this movement is at the Port of New York, through which approximately one-half of the foreign commerce of the country passes each year, and skirting the shores of which is a rapidly increasing, consuming population that now numbers over six millions of people. To serve these

eds, eight railways from all parts of the nation rminate at tidewater, from whence the principal rtion of their freight is floated to vessels and the congested and enormously expensive water rontage of Manhattan Island—a frontage that ould be devoted to the legitimate needs of ater-borne commerce.

Labor-saving devices for freight handling, espite the availability of cheap electric power, re practically unknown in the Port, partly due o doubt to lack of coordination between rail nd water interests. The question of an encircling outer belt line open to the use of all ailroads under conditions that will guard against iscrimination and excessive rates, is having no onsideration, although it is a crying necessity or interchange and manufacturing purposes nd, above all, for the preservation of the "open oor" to the Port. This last need requires special emphasis at this time in view of the rojected development of a municipal export erminal at Jamaica Bay, suitable Barge Canal erminals, and the Hackensack River develop ment, all of which will be largely worthless in he absence of an unrestricted outlet to all of he railroads that serve the Port. The deadening effect of the control by one railroad of access to any great industrial development, is well known.

As to any improvement of internal methods of distribution, the whole subject appears to be at a standstill awaiting the abatement of the fever for much needed rapid transit passenger facilities.

Summarizing, it seems fair to state that, from a freight transportation standpoint, the Port of New York, embracing Jersey City, Newark, Elizabeth, Bayonne, Perth Amboy, Hoboken and Weehawken, as well as Greater New York, instead of throwing aside local jealousies and adopting a broad policy that will bring the greatest good to the greatest number, is growing like "Topsy," with no comprehensive harbor or terminal policy. No private corporation or group of individuals can alone bring about the needed reform. The hope seems to lie in an agitation of the question in journals like "Engineering News," and before commercial organizations in the various communities that are tributary to the Port, with a view to starting a movement that will result in some definite action.

In this connection it may be of interest to quote from a recent report of the Montreal Harbor Commission, after a systematic study had been made of port conditions in this country and abroad.

(1) The ports that are doing the biggest business, and doing it efficiently, are the ports that have kept their facilities ahead of actual requirements.
(2) The ports that have remained stationary or lost in prestige have been those that neglected to provide facilities before business was forced to seek elsewhere the same facilities provided by rival terminals.
(3) Unity of authority, concentration of business, depth of water areas, and facilities for despatch of business are the prominent characteristics of successful port administration.
(4) The necessity of providing large and convenient storage areas where cargo may be collected and cared for.
(5) The lowest cost of handling cargo from the hold of the ship to consignee and vice versa, was found to be in a port where one authority controlled the entire operation, and where the transit sheds were three to five stories high.

Admitting that there is an urgent need for action to improve terminal conditions in the Port of New York, the question arises—in what way can this best be accomplished? The good results that Boston and vicinity have reaped from the organization of the Metropolitan Districts on matters of community interest, such as water supply, sewage and parks, prompts the thought that it may be feasible for New York and the neighboring cities on both sides of the Hudson, to have a "Metropolitan Freight Transportation District," embracing all parts of the Port. To determine upon the wisdom of organizing such a district and to secure concrete results, it would seem feasible to bring about the appointment, jointly by the States of New York and New Jersey, of an "Interstate Terminal Commission" composed of men thoroughly qualified to pass upon questions affecting the physical and business relations between rail and water carriers and the public, for the purpose of making recommendations for joint legislation after an investigation of harbor conditions in this country and abroad.

The Port of New York lies not altogether in New York, as its name implies, but likewise in the State of New Jersey, and it may safely be asserted that without joint action, any broadly effective improvement in the terminal conditions is hopeless.

Competitive ports are awake to the need of energetic measures to keep abreast of the times, and it would seem as if the Port of New York should not wait to do likewise until her supremacy is more seriously threatened.

The Cost of Electric Operation of Steam Railways.

One of the subjects to be reported upon at the International Railway Congress this year is the electrification of steam railways. In the "Bulletin of the International Railway Congress" for January, there appeared a long paper on this topic by Mr. George Gibbs, M. Am. Soc. M. E., who has had the chief responsibility for carrying out the electric traction work for the Pennsylvania and Long Island railroads.

In view of Mr. Gibbs' extended experience in steam railway work as well as in electrical work, he is among the best qualified engineers in the United States to speak authoritatively upon this subject. We therefore reprint below the concluding portion of Mr. Gibbs' paper, containing the sections dealing with the design of electric locomotives and those relating to the first cost and cost of operation of electrified steam railways.

It is particularly interesting to note Mr. Gibbs' comparison of the cost per car-mile with electric and with steam traction on the Long Island R. R and on the West Jersey & Seashore R. R. On the West Jersey & Seashore R. R. the cost of operation per electric car-mile was within two cents as much as the cost of steam operation. On the Long Island R. R., where a frequent train service with short trains is necessary, the electric system made a much better showing, the cost of operation per electric car-mile being 17.8 cts. as compared with 27.95 cts. for cars hauled by steam locomotives. These, however, are operating costs only and take no account of the interest charge on the electric equipment. If this were added to the cost of electric operation there can be little doubt that the steam locomotive would make the better showing.

Mr. Gibbs' paper follows:

ELECTRIC LOCOMOTIVES.—Electric locomotives for heavy service are still in the development stage; much has been done towards perfecting their design, but much remains to be done before the questions of their design and practical performance reach the stage of accumulated knowledge which we possess of these elements in steam locomotives.

In the earlier stages of the art, attention was mostly directed to the perfection of electric apparatus; the importance of the mechanical features of the machine in its adaptation to track conditions, were not fully appreciated. In fact, it was assumed that the characteristics of rotary motion and uniform torque, possessed by an electric motor, made its application to a locomotive a simple matter, and removed one of the chief defects of a steam locomotive, the unbalanced reciprocating weights and unsymmetrical turning effort, which were held to be destructive to track.

Experience has taught, however, that there are other elements in a locomotive which may be even more destructive, namely, unduly low center of gravity and improper weight and wheel-base distribution. Defective design in these respects becomes especially noticeable in a machine where power is applied with exraordinary compactness, and where the machinery for its development is naturally centered around the driving axles, and the weight concentrated below the wheel tops. It appears to be a fact that in present designs of electric locomotives, a given total weight and individual wheel weight is more destructive to track than the same weight distributed as in steam locomotives. It is noticeable in motor cars even, that the quite moderate axle weights of a motor truck have an unfavorable effect upon the track.

The importance, therefore, of a study of the electric locomotive problem from standpoints other than that of power development, is being recognized and the cooperation of practical railway men in the design is being sought.

The first electric locomotives were practically powerful motor cars; that is, single-reduction motors were mounted on bogie trucks and carried a body and cab for housing the control apparatus. Then steam locomotive design began to be copied in the use of rigid frames carrying at times four axles and motors, and gearless motors of large capacity and great weight were employed. In the greater number of the earlier designs all weight was adhesive for driving—a feature which was supposed to constitute one great advantage of an electric over a steam locomotive.

These heavy and powerful locomotives at high speed began to manifest their bad riding qualities, and they were suspected of being "hard on track." In order to bring out the facts experimentally, the Pennsylvania R. R. Co., who were about to design locomotives for their tunnel entrance into New York City, constructed a special test track with apparatus for measuring side pressures upon the rail; they built sample locomotives of different designs and instituted a series of tests of electric and steam locomotives to determine their relative riding qualities at speed.

It was found that all types of locomotives were practically steady at speeds under 40 miles per hour, but that above this speed marked differences appeared; that the steadiest riding machines were those with high center of gravity and with long and unsymmetrical wheel base. In other words, that the nearer steam locomotive design is approached in wheel arrangement, distribution of weight, height of center of gravity and ratio of spring-borne to under-spring weight, the less the side pressures registered on the rail head.

In addition to the excessive side pressures on the rail head, due to the oscillation and "nosing" of a low center of gravity machine, abnormal track effects may be caused by the vertical pounding due to a large non-spring-borne motor weight, or to weights with imperfect spring cushion. A remedy for all of these defects appears to mean a radical departure in the design of a high speed locomotive, both as to its running gear and as to the form of motor drive. It means a combination of driving and carrying wheels, an unsymmetrically disposed wheel base and the setting of the motors on the main frames above the axles. The extent of the departures in practice thought desirable for heavy high speed locomotives, is shown in the diagrams and photographs of the Pennsylvania locomotive.*

First Cost of Electrification.

It is difficult, in fact, practically impossible to give any general statement of the first cost of electrification which can be safely applied to individual cases. In the case of an entirely new line, built for traction by electricity, unit cost may be made to apply, but in the case of conversion of an existing steam line there are many collateral costs to be considered aside from those of electric equipment. In few cases will an electrified steam railroad be conducted precisely in the same manner as that previously done by steam; that is to say, multiple unit train service at frequent intervals will displace part or all of the infrequent locomotive train service; existing tracks, yards and terminals, stations, signaling, and telegraph lines must be changed to a greater or less extent to adapt the road to the new class of service; special rolling stock must in part be purchased and in places new real estate acquired.

It therefore results that each electrification must be dealt with as a separate and individual problem and cost figures for one situation will generally not apply to another, even if the other road is of similar character. Generally speaking, the cost of converting a steam railroad is very high under American conditions. Taking into account the changes in the physical property and equipment it will at times mean a doubling of the existing capitalization.

The high cost of conversion will act as a deterring factor upon additional electric construction until a sufficient number of installations are made to furnish evidence of the advantages and economy of electric traction; in fact, railway managers although watching closely, and with interest, the pioneer installations of electric operation on steam roads have not been anxious to duplicate experiments in advance of a demonstration of the results, and are awaiting this demonstration.

The systems available for heavy traction are the A. C.-D. C. and the straight A. C., either single phase or polyphase. Comparative first cost figures for these two systems would be interesting, but are available only from estimates for the reasons before given and because of the fact that the systems are not in the same stage of development; thus, figures of cost for A. C.-D. C. would be more exact than in the case of the single phase because figures for the latter system must be made on assumptions of present methods of application, and these methods are, it is thought, susceptible to important modifications.

Considering the electrical items of installation, the chief elements of cost may be grouped under three headings; the power-house, the transmission system from power-house to the cars, and the train equipment.

Of these three divisions, in general the power-house is the least expensive and the train equipment the most.

*See Engineering News, Nov. 11, 1909.

For illustration, and comparing figures for the two systems, an estimate may be given of the case of a trunk line road with dense suburban traffic conducted by multiple unit trains and with an important through business conducted by electric locomotives. The percentage of the total electrical cost carried by the three equipment divisions were as follows:

	A.C.-D.C. system.	A.-C. system (single phase).
Power-house	23%	25%
Transmission, power-house to cars	35%	22%
Train equipments	42%	53%
	100%	100%

These percentage figures are given to show the relative importance, as regards cost, of the three divisions for the two systems, and shows the larger proportion carried by the train equipments of the single phase system.

The actual money value of the various costs would not be useful without a detailed presentation of the case, but the important fact developed was that there was little difference figured for the total first cost of either system, in other words, as at present developed, it appears that for a dense suburban service conducted by multiple unit trains, or, in the case of locomotive haulage in terminals or over short lengths of a main line, there would be little difference in first cost between third rail A. C.-D. C system and the single-phase, alternating-current system; in fact, the figured differences are so small as to be within the limits of error in calculation. As however, the length of line increases and as the frequency of trains becomes less and the weight of the trains increases, the single-phase system becomes cheaper in comparison with the A. C.-D. C. and a substantial saving in first cost can be effected by the use of single phase on such lines. In fact, for long, trunk-line work and with the average density of traffic found on American railways, the first cost of the A. C.-D. C. system would certainly be prohibitive, whereas there will be found instances where this will not be the case with the alternating-current system. It is, of course, a matter of importance to determine the prospect of reducing first cost. This might be accomplished by lowering in price of apparatus, or by a radical change in system, or by both. There appears to be a reasonable prospect of some substantial reduction in the first cost of electric apparatus, as the market for it broadens, but the manufacturers do not promise very radical reductions. As to the prospect of a change to a system inherently simpler and cheaper than existing ones, it would be unprofitable to predict what the future may bring forth; a system which would eliminate the element of transmission, power-house to cars, would seem to open up a prospect of important simplification and cheapening, but such a system does not appear to be in process of development.

Cost of Operation.

This is a matter of the highest importance to railway officials, since the question of net yield is the foundation for an investment. High first cost may act as a deterrent in the rapid introduction of an improvement, but in the end a new system will prevail if the operating results, as measured by the net earnings, show a betterment. Under another heading, will be given a summation of the advantages of electric traction and a statement of its probable field, but it is here proposed to give such concrete examples of operating costs as are available to the writer. It is to be regretted that the list is not longer, but it includes, perhaps, the only cases in which important installations have been in complete operation for a long enough period to furnish comprehensive figures.

The installations referred to are those on the Long Island R. R. and the West Jersey & Seashore R. R., and figures are presented for two complete years' operation in each case.

Below are condensed tabular statements, taken from the complete monthly and yearly tables, of the figures which can be usefully given without lengthy explanation. They may be referred to as follows:

Table I. gives a concise statement of the comparative physical characteristics of the electric installations on the two roads, from which a general idea of the magnitude of the operation may be gathered.

Table II. gives average number of cars in trains and the average weight per car. It will be noted that the average weight per car changes slightly in the two years, due to a change in the ratio of the motor to trailer cars.

Table III. gives comparative figures for electrical quantities and efficiencies and is of interest in showing the distribution of the various losses in electrical energy from the power-house to the cars. The figures for power consumption per ton-mile and per car-mile represent fair averages for electric installations of the character in question and in a climate which requires a considerable consumption of energy in winter for heating the cars. Electric heating adds very largely to power consumption, as will be illustrated when it is stated that, in summer months, the watt-hours per ton-mile at substations on the West Jersey & Seashore R. R. were about 63, while in mid-winter months this figure rose to 100; the larger part of this difference is due to the additional energy required to light and heat the cars in winter. It is interesting to note also that the efficiency of the electric system from the power station to the point where the current leaves the substation for feeding into the third rail is from 70 to 80%, depending upon the loading during the various months; in the busy months, the efficiency will at times run as high as 85%. The losses from the substation to the cars in the direct-current feeder system cannot be accurately measured, but in the cases cited it is estimated to be between 5 and 10%. The figures given for "current used for other purposes" illustrate the considerable and useful consumption of power for running shop motors, lighting stations and operating the signal system.

Table. IV. gives the cost of generating current at the power-house and cost of the current when delivered at the cars. The former figures include all power-house operating and maintenance expenses, but no fixed charges, taxes or depreciation charges. The "delivered cost" includes the operating and maintenance cost for transmission lines, third rail, track bonding, and substations, and allowances for losses in transmission of the current and its conversion to direct current delivered at the car contact shoes. It should be noted, however, that the above figures do not include any maintenance expenses other than for electrical apparatus, and no portion of the general expenses of the railroad.

Table V. gives the monthly car mileage and cost per car-mile; the total yearly mileage and the average car-mile cost; and average yearly cost per 1,000 ton-miles moved. The cost per car-mile includes the following: The cost of power, maintenance of third rail and track bonding, maintenance of car bodies and trucks, maintenance of way and equipment expenses, conducting transportation and traffic expenses, general expenses, wages of motormen and train crews; in fact, all operating expenses of the railroad. In comparing the costs per car-mile, the difference in weight of cars, given in Table II., should be noted. It should furthermore be explained that in the case of the Long Island R. R., all trains may be termed "local" as the average length of the run on the so-called express service is only about 2½ miles, whereas on the West Jersey & Seashore R. R. about one-third of the total train-miles are made in cross-country express service with an average length of run of about 25 miles between stops, and the remaining service termed "local" averages 2½ miles between stops, or about the same as express service on the Long Island R. R. In other words, the Long Island installation may be considered purely local service in a densely populated district, whereas the West Jersey service approximates main line railway conditions with long runs.

Table VI. gives a list of the principal defects reported, and is of interest in showing which portions of train equipment require the largest amount of attention. The item for blown fuses appears excessively large, but it should be remembered that the purpose of these fuses is to guard against excessive current in the various critical portions of the equipment, and the fuses are purposely adjusted so as to be blown under exceptionally severe conditions; they are easily and quickly replaced by the train crews and cause little delay to service.

The excessive number of fuses blown on the Long Island R. R. in 1908, was partly due to the use of stock fuses which were found somewhat too small for the increased requirements due to heavy loads and increased severity of service in that year. The considerable number of flashovers of motors on the Long Island R. R., was also largely due to very severe local service. Breakage of contact shoes is almost entirely due to striking material left by trackmen in the path of the shoes, but occasionally shoes are broken in striking the approach blocks where the third rail is interrupted at street crossings. It will be noted that there were a large number of defective trolley poles reported on the West

TABLE I.—STATISTICS OF TWO STEAM RAILWA CONVERTED TO ELECTRIC TRACTION.

	Long Island R. R.		West Jersey Seashore R.
	1907.	1908.	1907-1908.
Route miles of track	37.6	43.9	74.6
Miles of single track	90.0	98.69	152.5
Miles of high tension pole line	41.0	42.0	69.59
Miles of high tension circuit (3-phase)	70.24	80.03	417.54
Miles of high tension circuit (1-phase)	14.1	9.7	None.
Miles of conduit	9.09	9.09	.....
High tension transmission voltage	11,000	11,000	33,000
Number of substations	6	6	8
No. of portable substations	2	2	None.
Capacity of substations	19,500	19,500	17,000
Cap. of portable substations	2,000	2,000	None.
Capacity of power-house	16,500	16,500	8,000
Number of motor cars	130	132	80
Number of trailer cars	84	82	None.
No. of baggage & mail cars	5	5	8

TABLE II.—WEIGHT OF CARS AND MAKE-UP O TRAINS.

	Long Island R. R.		West Jersey Seashore R. R	
	1907.	1908.	1907.	1908
Average cars per train	3.70	3.94	2.95	3.4
Average net tons per car.	37.8	36.5	47.9	47.4

Jersey & Seashore R. R.; this is due to the use of wheel trolleys and the ordinary type of cross-span overhead trolley wire; it illustrates, that while this type of construction was installed in the best manner, it is not suitable in practical service with trains of three or more cars operating at fairly high speed.

Table VII. classifies the principal detentions of all kinds due to electric equipment and gives the total train minutes of delay due to same. It also gives the totals reduced to figures of "car-miles per detention" and "car miles per minute of detention." In compiling this table, detentions due to collisions and derailments have been omitted, as having no direct bearing upon the behavior of electric equipment.

GENERAL REMARKS UPON TABLES.—As before stated, the two roads have been in operation for a sufficiently long time to rather definitely establish the behavior of all apparatus in practical service. There is no indication that in subsequent years of operation there will develop cause for extraordinary maintenance in any particular items due to the accumulative effect of service. It should be noted, however, that no account is taken of possible advances in the electric art, which might make the equipment obsolete; in other words, no "depreciation" account has been kept. In a rapidly advancing art, replacements on account of "change of type" are quite possible contingencies, but much difference of opinion exists upon this whole subject in America and its pros and cons cannot be entered into in this general report.

Of the total cost per car-mile, one-third is due to cost of supplying power to the car, including maintenance of all parts of the electric system on the car, and of this figure about one-half is the cost of the power alone. In both roads, the load conditions at the power-houses are not favorable, nor is the quantity of power turned out as large as it is expected to be in the near future, so that the item for cost of power at the car should come down considerably as the magnitude of the operation increases. Other items also in the total cost per car-mile, namely, general expenses of various kinds and wages of motormen and train crews, are largely affected by density of traffic and have a downward tendency.

The list of detentions looks quite formidable, but when analyzed the general result is found to be good. Motors, as before stated, have been brought to a high state of

TABLE III.—ENERGY CONSUMPTION.

	1907		1908	
	Long Island R. R.	West Jersey & Seashore R. R.	Long Island R. R.	West Jersey & Seashore R. R.
1. A. C. kilowatt-hours received at substations	20,341,826	.....	21,899,739	.....
2. Efficiency of H. T. transmission, %	96.2	.....	97.7	.....
3. D. C. kilowatt-hours delivered from substations	16,138,965	14,585,900	18,138,515	16,088,300
4. D. C. kilowatt-hours used for electric traction	13,466,995	13,530,659	16,201,962	14,780,145
5. D. C. kilowatt-hours used for other purposes	896,075	389,672	901,934	731,771
6. Kilowatt-hours lost in conversion from A. C. to D. C.	4,202,861	5,882,510	3,761,224	5,997,020
7. Efficiency from power-house to substation bus-bars	78.52%	72.15%	81.3%	73.8%
8. Total ton-miles (passenger cars)	141,541,106	184,615,535	180,129,860	192,472,541
9. Passenger car-miles	3,808,479	3,855,580	4,945,719	4,044,025
10. Passenger train-miles	1,021,102	1,305,663	1,251,877	1,172,894
11. Watt-hours per ton-mile at substations	95.2	73.29	90.0	76.8
12. Watt-hours per ton-mile at power-house	121.2	101.58	110.6	104.0
13. Kilowatt-hours per car-mile at power-house	4.51	4.86	4.028	4.95
14. Kilowatt-hours per car-mile at substation	3.54	3.51	3.276	3.66

TABLE IV.—CURRENT AND MAINTENANCE COSTS.

	Long Island R. R.		West Jersey & Seashore R. R.	
	1907.	1908.	1907.	1908.
Cost per kilowatt-hour at power-house	0.804 cts.	0.697 cts.	0.680 cts.	0.592 cts.
Cost per kilowatt-hour at cars	1.705 cts.	1.461 cts.	1.302 cts.	1.151 cts.
Cost per car-mile for maintenance of electrical equipment on cars	0.71 cts.	0.76 cts.	.....	.....
Net output in kilowatt-hours at power-house	.....	31,517,200	.....	22,887,800
Pounds of coal per kilowatt-hour	.....	3.29	.....	3.36
Cost of coal per ton: 2,000 lbs.	.....	$2.51	.....	$2.18
Number and capacity of power-house units	.....	3—5,500 kilow.	.....	4—2,000 kilow.

TABLE V.—CAR-MILE COSTS.

Month.	Car-miles (1 = 1,000). West Jersey & Seashore R. R. 1907.	1908.	Long Island R. R. 1907.	1908.	Cost per car-mile, cts. West Jersey & Seashore R. R. 1907. Cents.	1908. Cents.	Long Island R. R. 1907. Cents.	1908. Cents.
January	294	281	235	201	27.50	29.29	28.16	26.97
February	255	271	214	186	30.90	28.53	30.04	27.87
March	270	291	241	224	23.90	26.57	28.05	24.77
April	244	331	281	271	24.30	21.34	24.05	23.94
May	267	323	347	382	21.50	19.97	21.39	20.90
June	292	344	349	478	22.50	18.92	22.32	18.00
July	435	451	392	628	15.30	16.17	15.73	13.53
August	478	485	400	632	14.90	13.71	15.59	13.26
September	411	369	299	509	15.85	17.17	19.87	14.79
October	317	312	306	441	23.50	19.55	22.37	17.21
November	292	282	245	345	21.40	20.42	25.93	22.29
December	295	292	212	360	22.60	22.68	27.48	22.45
Total	3,854	4,044	3,526	4,662	21.30	20.46	22.45	17.80
Cost per 1,000 ton-miles	....	....	....	....	$4.406	$4.307	$5.94	$4.86

TABLE VII.—DELAYS DUE TO ELECTRIC EQUIPMENT.

	1907 Long Island R. R. No.	Train Minutes.	1907 West Jersey & Seashore R. R. No.	Train. Minutes.	1908 Long Island R. R. No.	Train. Minutes.	1908 West Jersey & Seashore R. R. No.	Train. Minutes.
Motors	13	197	21	404	35	270	20	134
Control equipment	21	286	180	2,209	72	644	99	509
Air brakes	8	84	40	247	38	428	44	54
Miscellaneous mechanical equipment	7	74	80	1,073	57	964	45	427
Miscellaneous electrical equipment	58	1,006	316	1,564	44	781	183	693
Failures of power due to 3d rail	17	218	8	103	36	792	3	47
Failures of power due to trolley	..	...	86	1,122	...	...	223	3,498
Failures of power due to S.-S. & Trans.	5	25	13	80	31	190	36	284
Track troubles	7	263	7	146	1	15	1	18
Snow and ice on 3d rail	8	161	72	1,751	16	244	9	69
Shoe fuses blown	..	...	...	...	107	1,881	...	...
Unclassified and unknown	15	179	120	844	53	552	27	108
Total	159	2,493	943	9,543	490	6,761	661	5,841
Car-miles (passenger & baggage)		3,526,227		3,855,580		4,662,230		4,044,025
Car-miles per detention		22,177		4,088		9,514		6,118
Car-miles per minute of detention		1,414		404		689		692

perfection and detentions caused by their failure on the road are rare and largely due to overloading. The temptation in electric service is constantly to overload, either quickening the schedule, increasing the number of stops or adding trailer cars; this is due to the fact that the apparent limit of capacity of railway motors is their ability to make schedule and it is often not remembered by the railway superintendent that this is far beyond the heating capacity of the motors.

Where all cars in a train are motor cars, overloading cannot be as readily accomplished as where trailers are hauled, especially if the motors have been designed to take care of continuous running conditions with the minimum number of stops, as is generally the case.

The multiple unit control system is a very complicated assemblage of delicate apparatus, but works surprisingly well and, properly maintained, gives little trouble.

Table VI. is confined to the statement of delays and troubles on the train equipment; but in Table VII. the troubles due to the other parts of the electric installation, namely, the third rail, transmission lines, substations and power-house, are included and it is interesting to note the unavoidable delay caused by these features of an electric installation.

Cost Compared With Steam.

Considering the figures for the year 1908, it is seen that the Long Island road operated their electric mileage at a cost of 17.80 cts. per car-mile; the steam train mileage cost 27.95 cts., a difference in favor of electric traction of 10.15 cts. per car-mile. On the West Jersey & Seashore R. R., the electric mileage cost 20.46 cts. per car-mile as against 22.30 cts. for the steam mileage, or a saving for the electric service of 1.94 cts. per car-mile.

The relative unfavorable showing on the West Jersey for electric saving is in part accounted for by the difference in character of the service on the two roads, as before explained, and by the fact on the Long Island road the average length of train in steam service was about the same as in the electric and the stops frequent; it was, therefore, costly service. On the West Jersey the average cars per steam train was twice that of the electric and much of the service was long distance express with few stops and of an exceptionally economical character.

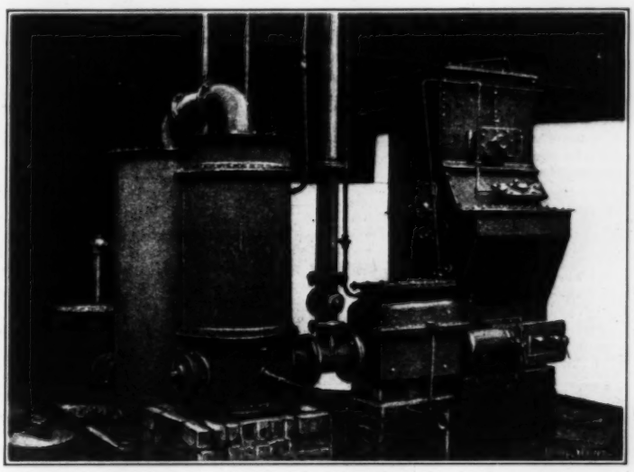
TABLE VI.—DEFECTS OF TRAIN EQUIPMENT

	1907 Long Island R. R.	West Jersey & Seashore R. R.	1908 Long Island R.R.	West Jersey & Seashore R. R.
Flash-overs	23	None	42	1
Main fuses blown	50	..	60	..
Shoe fuses blown	1,169	844	4,812	1,934
Trolley fuses blown	None	358	None	711
Bus fuses blown	158	51	182	169
Controller fuses blown	None	138	None	118
Heater & Pump fuses blown	379	359	590	383
Total fuses blown	2,023	1,955	6,028	3,504
Hot motor axle bearings	6	8	25	2
Hot journal bearings	6	44	34	28
Grounded armatures	9	6	27	6
Short circuited armatures	1	In above	9	In above
Grounded fields	1	2	2	1
Commutators	5	None	16	None
Contact shoes replaced	500	925	672	564
Pump motor armatures	10	1	15	2
Control	86	180	139	99
Brake equipment	15	40	28	15
Short circuits (misc.)	5	..	9	..
Trolley poles	Not used	981	Not used	489

Operating Costs; A. C.- D. C. vs. A.C.

A comparison of this kind must at present writing be based on estimates and opinion, as no operating figures for the A. C. systems are obtainable. It is the opinion of the writer that the maintenance costs of the single-phase system, as at present developed, will be somewhat higher than for the A. C.-D. C., but not by an important amount; eventually the maintenance costs should be about the same for either system. The total operating cost figure, however, will probably be in favor of the single phase system, because of the higher average efficiency of this system and the lower operating cost of substations. The importance of the savings possible for the single phase will depend upon the character and extent of the traction scheme, becoming greater as the length of line increases and the density of traffic decreases. It is impossible, however, to generalize safely and each case should be considered upon its own merits.

An estimate may be given for the particular case referred to under the "first cost" heading, in which it was figured that about 13% less energy at the power-house would be required for the single phase than for the A. C.-D. C. system. This, together with the saving in substation operation, would show an operating saving of about 1 ct. per car-mile, or, say, between 4 and 5% of the total operating cost.



AN ENGLISH BITUMINOUS-FUEL SUCTION GAS PRODUCER.
Designed by Mr. G. L. Morton, Birmingham, England.

A Bituminous-fuel Suction Gas Producer.

A suction gas producer, in which ordinary cheap coals can be satisfactorily used, has been sought by many designers of gas power-plant apparatus without notable success. One such producer, designed by Mr. Geo. L. Morton, Birmingham, England, is reported to have successfully passed many severe tests. This producer is described in the January issue of "Engineering Review" (London), from which the following items are taken:

The Morton plant is as simple as an ordinary anthracite suction plant, consisting of a generator, steam raiser, cooler and scrubber.

The generator, which is the subject-matter of the patents, may best be described as a rectangular shaft, having on the front side a projecting space; this projection slopes downwards, the angle of inclination approximating to the angle of repose of the face of a heap of fuel. From the outer end of this projection a return slope connects with the shaft near the base.

The generator is of the down-draft type, and is charged with fuel from the top through an opening provided with a movable lid. The top part of the shaft is a fuel container. Just below the commencement of the projection, but on the back and therefore opposite, is placed the main air inlet to which a pipe from hand-driven blower is attached. The projecting space is long and narrow, and on each side of the space are secondary air inlets, whilst at the angle formed by the meeting of the two slopes is a further air inlet.

At the base of the shaft is a grate with ordinary fire bars. Below is an ash container with airtight door for removal of ash, etc. On a level with the fire bars on the front and under the return slope is a chamber, closed by an air-tight door, for raking out the fire; this chamber projects beyond the generator casing, and on the top of this is the gas-pipe leading to the cooler and scrubber, which are connected with the expansion box and engine.

The generator is started cold, by charging in broken gas coke up to the level of the air inlet from the blower. A fire is kindled on this and coke is charged until the container is full; all air inlets are closed; the blower is worked and the fuel is blown up to incandescence. As soon as combustible gas is formed the engine is started, the main inlets opened and steam is admitted. The generator is now working by the suction stroke of the gas engine. As soon as the engine is fully at work the two side air inlets are opened and steam is admitted. When the coke in container has fallen sufficiently, raw coal is charged in, and the air inlet at meeting of the slopes is opened and steam is admitted.

The coal in the container, resting on the incandescent coke bed, gives off its volatile matter and continues doing so until reduced to coke. This volatile matter, as gas, is drawn down to the gas outlet and follows the easiest path. This is provided by the sloping cover to the producer, for here the fuel cannot pack. The fuel along this slope is kept incandescent by the side air inlets and the volatile matter passing through the mass is reduced to a permanent gas. Free carbon is deposited but largely burned by the air admitted through the side openings. CO_2 is also converted to CO.

The container is always full of fuel above the commencement of the slope and as the lower part of the bed sinks fresh fuel rolls down the slope and no open spaces are formed where gases can collect and burn.

At night the generator is banked full of coal and is started up next morning by using the hand blower. From the time blowing commences until the engine is on a working load is only six minutes.

The engine has been run for 1½ hours with no load and then 45 B. HP. (94% full load) has been thrown on suddenly without slowing down the engine which within two minutes was cutting down the supply. After running at quarter load for 1¼ hours full load was thrown on quickly without slowing down. This engine was a "Dudbridge" type, rated by the maker at 55 B. HP. on city gas and at 42 B. HP. on producer gas. On gas from the Morton plant, 48 B. HP. was carried.

The consumption of an ordinary Warwickshire coal, containing about 10 gals. of tar per ton, is given as 1 lb. per B. HP.-hr. The gas seems to be of uniform quality, costing 36 cts. per 1,000 cu. ft. or one-fourth as much as city gas.

A Comparison of Methods of Computing the Strength of Flat Reinforced-Concrete Plates.*

By ANGUS B. MACMILLAN.†

One of the most recent developments of reinforced concrete is the type of floor originating with Mr. C. A. P. Turner and known as the "Mushroom Floor," such a construction, for instance, as is shown in Fig. 6. While this system is in its infancy as yet, examples of it are nevertheless sufficiently numerous to be more or less familiar to all building designers. The absence of beams, the increase of headroom resulting therefrom, the low cost of centering, and the comparatively small amount and simplicity of the steel reinforcement, unite to make this system desirable for many buildings and relatively low in cost.

With its advantages, however, it has some disadvantages, perhaps the most important of which is a lack of detailed analysis of the internal stress occurring in a slab of this kind. It may be mentioned that this analysis is not necessary for safe construction, for as the more or less empirical formulas we have seem to give results sufficiently strong, enough confidence can certainly be placed in the formulas to enable the designer to guarantee a test load of twice the designed live load over an entire bay without harm to the construction and with a very insignificant amount of deflection.

About two years ago, the writer had occasion to design floors of this kind for an office building in Portland, Me. At that time the amount of information concerning the stresses in the concrete plate was very limited, and the writer had some difficulty in finding a satisfactory solution.

A good deal of study of buildings designed by the inventor gave the impression that Mr. Turner used a formula equivalent to about $M = \frac{WL}{48}$ to get a bending moment, and that he then solved for an area of steel and distributed the resulting amount uniformly or nearly so, in the different directions. This seemed so extreme, however, that no confidence was placed in the result and various other ways of solving the problem were tried.

For purposes of comparison the writer proposes to give a description of the different methods then used or that have since come to his knowledge, together with the solution by each of a typical floor panel. The methods are as follows:

1. To consider the slab around the head of the column out to the point of inflection as a cantilever supporting a uniform load over its surface and a concentrated load consisting of the rest of the slab hung from its periphery. This will be called the "Cantilever System." (Fig. 5.)

2. To consider the slab around the head of the column out to the point of inflection as a flat circular plate supporting a uniform load over its surface and a concentrated load consisting of the rest of the slab with its superimposed load hung from the perimeter of the plate. Diagrams for the analysis of this method are given in Turneaure and Maurer's "Principles of Reinforced Concrete, pp. 272 and 328. (Fig. 4.)

The first impression of these two methods is that they are almost identical, but the solution and final results are quite different.

3. Grashof. In Green's "Structural Mechanics," on p. 255, is given the formula for deflection and bending moments in a plate supported on rows of points forming squares whose side is L. A direct application of this to the typical case will be tried. Two other methods result from different application of these formulas; that by Mr. L. J. Mensch and the one by the writer.

4. Mensch. A more or less incomplete description of this method is given by Mr. Mensch in his "Reinforced Concrete Pocket Book," on p. 81, as follows:‡

> The problem of calculating the stresses in a plate supported at four points was first thoroughly investigated by Mr. Grashof in connection with the strength of end plates in steam boilers. He gives the greatest bending moment per lin. ft. in such plate as $\frac{WL^2}{26.5}$, which formula he derived more by an elimination process than by exact science.
>
> By similar reasoning, we lay down the following rule for the computation of girderless floor construction: Divide the panels in strips of a width equal to $0.35L$ when L is the distance c. to c. of support in feet; two strips run diagonally while the other run in the line of the columns. The greatest bending moment of such a strip we assume to be $\frac{WL^2}{20}$ when the size of the capital is at least $0.23L$. From this bending moment, we easily obtain the required thickness and reinforcement given in the slab tables.
>
> In the solution of the typical case the writer has simply copied the slab thickness and amount of steel directly from the tables mentioned, and has not endeavored to perform the actual necessary operations.

5. Turner. Turner confidently recommends and uses a formula that requires considerably less steel than any other method given here. He refers to Grashof's analysis of the stresses in plates given in Lanza's "Applied Mechanics," and to Prof. H. T. Eddy's "Theory of the Flexure of a Thin Flat Circular Plate or Ring Loaded Symmetrically About Its Center," Engineering Year Book, Univ. of Minn., 1899.

In his book recently published on the subject, he states that the bending moment $M = \frac{WL}{50}$ where W is the total load on the slab, and L is the span in feet or inches according as the result is desired in foot or inch-pounds. Having obtained the bending moment and assumed a depth for the slab, the area of steel is found. This area is divided in the different directions and placed in bands about 7/16 of the span in width.

The method of getting the stress in the radial bars and circular rings is left in doubt. The writer has assumed that they have no function other than to keep the slab steel up near the top of the slab where it belongs when crossing the columns, and to a very limited extent to give a larger bearing for the concrete plate.

When fixed, the size of the column heads is not stated to the writer's knowledge.

Turner supports his conclusions with the records of many slabs tested with loads largely in excess of the regular working loads.

6. MacMillan. This method is based on general deductions made from Grashof's analysis of the stresses in a plate supported at rows of points forming squares, whose side = L. Before taking up a typical case it might be well to indicate what these deductions were. Attention is called to the following formulas giving the maximum fiber stress in square plates uniformly loaded but with different methods of support.

a. For a square plate supported on two edges, $f = \frac{3WL^2}{4T^2}$

b. For a square plate supported on two edges, $f = \frac{WL^2}{2T^2}$

c. For a square plate fixed on four edges $f = \frac{WL^2}{4T^2}$

d. For a square plate fixed on four points, $f = \frac{WL^2}{4.27T^2}$

It is interesting to note that a plate fixed at four points is stronger than one fixed on four sides, and is 3.2 times as strong as one supported on two edges.

Now in the first formula the Moment of Resistance can be expressed by $M_r = \frac{WBL^2}{8}$

Therefore for a plate fixed at four points and the same unit load, the bending moment would be $\frac{1}{3.2}$ times this amount, or $M_r = \frac{WBL^2}{8 \times 3.2} = \frac{WBL^2}{25.6}$

Using 25 instead of 25.6 for a denominator, the result will be $M_r = \frac{WBL^2}{25}$ a formula that expresses the maximum stress over the head of the column.

Out in the center of the slab, the stress would actually be about one-half of this, or $\frac{WBL^2}{50}$. However, while in continuous slabs supported on beams making the different spans equal, the bending moment at the center of the span, $M_r = \frac{WL^2}{24}$, the different city building laws only allow the use of $M_r = \frac{WL^2}{10}$ or at most $M_r = \frac{WL^2}{12}$.

So in this type of floor, the writer sees no good reason for departing from a standard practice and has used the maximum moment and neglected the lesser one.

Actually, as all bars are lapped over the columns, there is a large excess of steel at this point, but as the extra strength developed by the lapped bars is to some extent uncertain, it is probably better to get an excess than otherwise.

Having determined the bending moment, it is a simple matter to get a steel area. This steel area is divided up into a number of small bars and distributed in uniform bands in the different directions.

It might be well to say at this point that two other distinct methods were tried without satisfactory results. One depended on considering the amount of load each separate bar should carry, allowing a certain amount of sag in the bar and figuring the stresses in the rod as a catenary. The other was to replace the straight bars from column to column by an imaginary beam on all sides of the panel as per sketch, Fig. 1, and to consider that the bending moment in plane $X - X$ was caused by the loading indicated by the shaded portions of slab; and that it was equal to

$$\frac{\frac{W.}{2} L}{12}$$

where W equals total load on panel, and L equals span. The steel to take up this stress was then obtained in the ordinary way, except that the diagonal bars were considered to take a stress equal to 16,000 $\sin \phi$ and to be equally effective in resisting stresses in a plane $Y - Y$ If the beams had really existed, this might have been permissible, but as they did not, it seemed that two other types of loading were possible, one giving

$$M = \frac{\frac{W.}{2} L}{8}$$

and the other

$$M = \frac{\frac{W.}{2} L}{6}$$

just double that originally considered.

Of course, if the slab was considered continuous, these moments would in every case be cut down. However, the conditions were not satisfactory, and taking the most unfavorable method of loading, the amount of steel necessary was excessive.

The symbols that will be used in analyzing the different cases are as follows:

- L = the span.
- B = breadth of panel.
- W_1 = live load.
- W^2 = dead load.
- $W = w_1 + w^2$ = combined load.
- M = bending moment.
- R = reaction at end of strip.
- I = moment of inertia.
- Δ = deflection.
- T = total thickness of slab.
- c = ½T.
- E_s = modulus of elasticity of steel in tension.
- E_c = modulus of elasticity of concrete in compression.
- f_s = unit fiber stress in steel in tension.
- f_c = unit fiber stress in concrete in compression.
- d = depth from top of slab to center of steel.
- jd = arm from center of compression to center of tension = $0.86d$.
- b = width of strip considered.
- A_s = area of steel.
- Σ = summation.
- V = fiber stress in concrete in shear.
- a = span in inches in "Grashof" method.
- r = radius of column head. } Turneaure and
- r_o = radius of point of inflection. } Maurer method.

In working out the amount of steel in the different designs, the same unit stress, i. e., 16,000 lbs. per sq. in., has been used in every case, and in Turner's method the slab has also been worked out for the regular working stress of 13,000 lbs. per sq. in. used by Turner.

In Turner's, Grashof's, and the writer's methods, the unit stress will be less than that adopted for the "Cantilever" and "Turneaure and Maurer," as it was not considered advisable to make the slab less than eight inches thick owing to difficulties arising in covering up the steel at the columns in thinner slabs.

In getting the deflection in the method called "Grashof" the deflection was assumed to depend directly on the modulus of elasticity of the concrete, which for stresses up to about 600 lbs. per sq. in. has been taken at 3,000,-

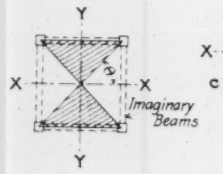


Fig. 1.

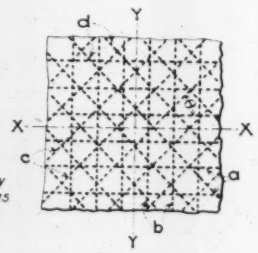


Fig. 2.

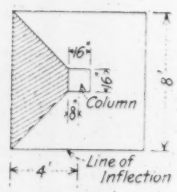


Fig. 3.

*Paper read before the Annual Meeting of the National Association of Cement Users, Chicago, Ill., Feb. 21-25, 1910.

†Engineer, Aberthaw Construction Co., Boston, Mass.

‡Notation changed to make uniform with the rest of this analysis.

900 as recommended by Turneaure and Maurer in their article on deflections.

For a typical case to be solved by each method, let the span = 20 ft.

Live load = 200 lbs. per sq. ft.

Panel to be square.

1. Cantilever Method.

Let L = span = 20 ft.

Let w = live load = 200 lbs. per sq. ft.

Assume slab 8 ins. thick,

Then w_1 = dead load = 100 lbs. per sq. ft.

Columns 16 ins. sq.

$W = w + w_1 = 200 + 100 = 300$ lbs. per sq. ft.

Point of inflection of slab for uniform load = $\frac{L}{5} = \frac{20}{5} = 4$.

Lay out square with half diameter of 4 ft. around column as per diagram (Fig. 3).

Find moment at side of column of portion, indicated by shading = 4,667 lbs.

Load on shaded portion at 300 lbs. per sq. ft.

Center of gravity of area = 24¾ ins. from column edge.

$M = 4{,}667 \times 24\frac{3}{4} = 115{,}500$ in.-lbs.

We must add to this the moment of the concentrated load hung from the perimeter of the square. This is equal to the load on the entire bay minus the load on the square under consideration divided by the number of sides of the square times a lever arm.

Lever arm = 40 ins.

$M^2 = 25{,}200 \times 40 = 1{,}008{,}000$ in.-lbs.

$M_1 = M_2 = 115{,}500 + 1{,}008{,}000 = 1{,}123{,}500$ in.-lbs.

Under the straight line theory:

for maximum compression in concrete = 750 lbs. per sq. in., and

for maximum tension in steel = 16,000 lbs. per sq. in., $\frac{E_s}{E_c} = 15$ $M = 133\ bd^2$ and $A_s = .0097\ bd$. Then taking b = 16 ins. = width of column side, and solving, d = 23 ins., say 27 ins., from top of slab to point where column head starts to flare out, and the area of steel at this point = 3.56 sq. in.

It is advisable to obtain the moments and thickness of concrete required to resist the stresses found at several points between the side of the column and the line of inflection of the slab. With these additional points, it will be quite easy to plot a curve for the column head. If these intermediate points are not located, it will be quite safe to carry the column head up at an angle of 45° with the horizontal, as in this instance. Later, when the amount of steel in the slab is obtained, see if it equals the area needed. If not, it will be necessary to add some extra bars over the column head.

Now consider the stresses in the diagonal bands. Take a strip (Fig. 5), 12 ins. wide between the points of inflection. As the bars cross in the center assume that each band of steel takes one-half the full load where they cross and the entire load where they do not cross.

As the load is uniform the reactions at each end are equal, and R = 1,950 lbs., M = 6,938 ft.-lbs., = 83,256 in.-lbs., and (by straight line theory) d = 7.21 ins., and area of steel = 0.8392 sq. ins. If ⅝-in. round bars are used, the number of bars required will equal 2.73 in strip one foot wide. Then the spacing of bars in the diagonal bands will equal $\frac{12}{2.7} = 4.4$ ins. on centers. The bands are to be made 8 ft. wide, as this fills the area.

Next, the stresses in the straight bands between lines of inflection will be obtained. These may be arrived at in a similar way. Take a strip 12 ins. wide. Over a portion of the length there is but one line of bars, so it may be considered that the whole load is acting here. Near the points of inflection, however, the diagonal bars cross and as there are three layers of bars, we assume that each line takes but one-third of the load at these points. On this basis, the placing of the steel being symmetrical and the loads uniform, the reactions of the slab at each end are the same and M = 4,904 ft.-lbs. = 58,848 in.-lbs. Using d as obtained for diagonal strip = 7.21 ins. and with $jd = .86d$, then $A_s = 0.594$ sq. ins., which can be obtained with ⅝-in. round bars, 6.4 ins. on centers. The bands are to be 8 ft. wide as before. The total area of bars, then, that cross the column cap is equal to $.594 \times 8 \times 2 + .839 \times 8 \times 2 = 22.9$ sq. ins. As the bars, if continuous, would resist stresses in opposite sides of column head, we assume one-half the total area as effective for taking care of the stresses in any one side of the head.

Now, $\frac{22.9}{2} = 11.45$ sq. ins.; but from the previous calculations only 3.56 sq. ins. were found necessary, so there is a large excess of steel.

Actually this excess would be still greater, since in practice the bars seldom come along enough to be carried continuously over more than one span, especially in the diagonal direction, so a lap is necessary at the columns. The usual method is to schedule the bars long enough to extend two feet beyond the center of the column at each end, making a lap of four feet.

2. Turneaure and Maurer Method.

A careful study of the diagrams given in Turneaure & Maurer's "Principles of Reinforced Concrete Construction," pp. 290-291, would indicate that as the ratio of r to r_o decreases, the bending moments in the circular plate decrease. This being the case, it would seem to be an advantage to have a fairly large and very stiff column head. This is readily obtained with the common form of capital used in this type of floor. For instance, assume a circular column 16 ins. in diameter. The column head has a flare of about 12 in. and a depth of about 30 ins., making a total diameter of about 40 ins. at the top. The ratio of depth to flare is 2½ to 1, which should ensure ample stiffness.

To take the case used in the preceding examples: Span 20 ft. and live load of 200 lbs. per sq. ft., it will be necessary to assume the slab somewhat thicker than in the other cases in order to get enough material to resist the apparent bending moment. It might, perhaps, be admissible to thicken the slab gradually near the columns, but Turneaure and Maurer do not state this; and further, it might complicate the centering to an extent that would not be economical.

Assume the slab 12-ins. thick. Taking concrete at 150 lbs. per cu. ft., the slab weighs 150 lbs. per sq. ft. of area. This, added to the live load, makes a total of 350 lbs. per sq. ft. With column spacing 20 ft. apart, the diagonals will be 28.28 ft. c. to c.

As the point of inflection of a beam fixed at the ends and continuously loaded is about one-fifth of the span length from the end, take the mean for the straight length and diagonal. This will equal approximately

$$\frac{\frac{20}{5} + \frac{28.28}{5}}{2} = 4.83, \text{ say about 5 ft.}$$

Having assumed a line of inflection, the area enclosed may be treated as a circular plate having a uniform load over its area and a concentrated load hung from its periphery equal to the remainder of the panel load (Fig. 4.) This will equal $\frac{(20 \times 20) - (5 \times 5 \times 3.1416) \times 350}{10 \times 3.1416}$ = 3,580 lbs. per lin. ft.

The ratio of the column head to the diameter of circle assumed $\frac{r}{r_o}$ is as 1 to 3.

From the diagrams on pp. 290-1, Turneaure & Maurer's book it is found that the moments caused by the uniform load on circular plate are:

1. Circumferential at column = 140 ft. lbs. per ft. width.

2. Radial at column = 1,225 ft.-lbs. per ft. width. And for the concentrated load at the edge are:

3. Circumferential at column = 1,790 ft.-lbs. per ft. width.

4. Radial at column = 14,320 ft.-lbs. per ft. width. Adding the moments together, we have = 17,475 ft.-lbs. = 209,700 in.-lbs. per ft. width.

Under the same stresses, etc., used in the Cantilever System (Method 1) $d = \frac{209{,}700}{133 \times 12} = 11.4$ ins., a depth that does not allow very much room to cover up the steel with the thickness of slab assumed.

Assume that this stress is constant for every linear foot of circumference of cap. The circumference of the capital being equal to 125.66 ins., the area of steel needed around the column head = $125.66 \times 11.4 \times .0097 = 13.90$ sq. ins.; but as the bars are continuous, only one-half of this amount is needed, = 6.95 sq. in.

These bars are to be arranged in four bands over the head of the column, and the results will probably be near enough if the amount of steel (6.95 sq. in.) is divided by 4 = 1.74 sq. in. in band the width of the column head: $\frac{1.74}{40} = .0435$ sq. in. per in. width.

Using ⅝-in. round bars with an area of .3068 sq. in., the spacing will be = 7.07 ins. on centers. This spacing had better be continued out to the point of inflection in each case, making the bands 10 ft. wide = 17 ⅝-in. round bars per band. It may be that some of these bands can be omitted in the slab between the points of inflection.

Consider a diagonal strip first: Referring to Fig. 4, noting the small figures that indicate the numbers of bars crossing, and assuming that each set of bars takes its proportion of the load, the loads are, scaling the distances $\frac{2 \times 2 \times 350}{3} + \frac{2 \times 2 \times 350}{1} + \frac{10 \times 350}{2} = 3{,}616$ lbs. The reaction at either end = $\frac{3{,}616}{2} = 1{,}808$ lbs.

$M = 1{,}808 \times 9 - 875 \times 2.5 - 700 \times 6 - 233 \times 8 = 8{,}020$ ft.-lbs. $8{,}020 \times 12 = 96{,}240$ in.-lbs.

Taking depth to center of steel area as 10½ ins., it will be found that $\frac{96{,}240}{.86 \times 10.5 \times 16{,}000} = 0.667$ sq. in.

is needed and $\frac{0.3068 \times 12}{.667} = 5.52$ ins. on centers

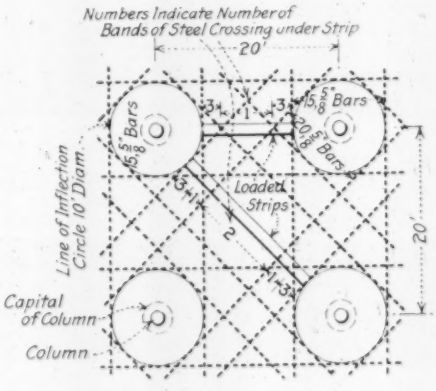


Fig. 6. Method No 2, Manner of Loading.

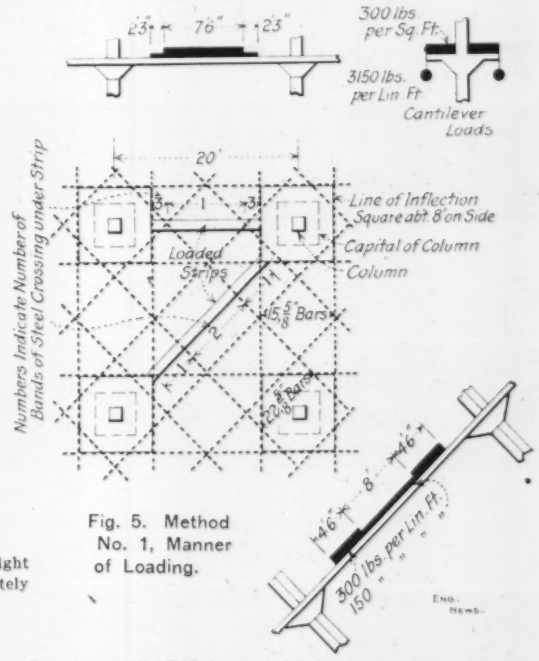


Fig. 5. Method No. 1, Manner of Loading.

against the 7.07 ins. previously found. It would seem advisable to space the bars somewhat closer together in the diagonal directions, say 6 ins. on centers, making 20 bars in a 10-ft. band. This ought to allow of spacing the remaining bars somewhat farther apart.

The load in the straight direction on a unit strip equals $\frac{2 \times 2 \times 350}{3} + 6 \times 350 = 2{,}566$ lbs.

The reaction at each end equals $\frac{2{,}566}{2} = 1{,}283$ lbs.

$M = 1{,}283 \times 5 - 1{,}050 \times 1.5 - 2.32 \times 4 = 3{,}908$ ft.-lbs. = 46,896 in.-lbs.

$\frac{46{,}896}{.86 \times 10.5 \times 16{,}000} = .325$ sq. ins.

$\frac{.3068 \times 12}{.325} = 11.35$ ins. on centers for spacing of ⅝-in. round bars.

As the stresses in the bars increase toward the edges of the bands, a spacing of about eight inches on centers is, perhaps, all that is advisable. That is equal to 15 bars in a band.

3. Grashof's Analysis.

Grashof's analysis of the stresses in a plate supported at rows of points making squares whose side is a

$$f = \frac{15\ W a^2}{64\ T^2}; \text{ and } \Delta = \frac{15}{512} \frac{W a^4}{E T^3}$$

As these formulas are worked out for steel plates for which Poisson's ratio is somewhat different than that for concrete, the results obtained may not be accurate, but it will be interesting to make the comparison.

Let L = the span = 20 ft.

Total thickness of slab = 8 ins.; dead load = 100 lbs. per sq. ft.

Live load = 200 lbs. per sq. ft.

W = total load per sq. in. = 2.08 lbs. per sq. in.

a = length of side in inches = 240 ins.
T = effective thickness of slab = 7 ins.

$$f = \frac{15 \times 2.08 \times 240 \times 240}{64 \times 7 \times 7} = 573 \text{ lbs. per. sq. in. maximum fiber stress.}$$

However, this fiber stress is based on the moment of inertia of a homogeneous material like steel, whose center of gravity is at half the depth.

As this does not hold for concrete, the bending moment per unit width may be determined by the formula

$$M = \frac{fI}{c} = \frac{573 \times 7 \times 7 \times 7}{3.5} = 56{,}162 \text{ in.-lbs. per ft. width.}$$

A somewhat lower stress on the concrete than in the previous example will be used. In this instance $f_c = 650$ lbs. per sq. in.

$f_s = 16{,}000$ lbs. per sq. in.

$$d = \frac{M}{108\,b} \text{ and } A_s = .0077 \text{ bd.}$$

From those formulas we get

$$d = \frac{56162}{108 \times 12} = 6.59 \text{ ins., which is fairly near the 7 ins. we assumed for } d, \text{ and}$$

$A_s = .0077 \times 6.59 \times 12 = .608$ sq. ins. per ft. width.

From a comparison with the position and magnitude of stresses in continuous beams, it will be safe to consider the maximum stress in the present case is the negative stress at the column in the direction of the diagonals, and that the positive stress half-way between the columns diagonally is but one-half this amount or less.

Also assume that the concrete plate will get the full stress indicated at the column and must be able to resist this stress from any direction, so that instead of one set of reinforcing bars, there must be at least two sets crossing at right angles to each other. Actually in this system there are four sets which will be lettered a, b, c, and d, crossing as shown in Fig. 2. It will now be apparent that any stress tending to rupture the slab in the vertical plane X-X, will be resisted by bars b directly up to their area bA_s, that and that bars c and d will assist in the ratio of $A_s \cos \phi$. Then to resist the stress in plane X-X.

$f_s\ (bA_s + cA_s \cos \phi + dA_s \cos \phi)$
$\phi = 45°$, $\cos \phi = .707$

Now as the component forces of bars c and d for resisting stresses in the vertical plane Y-Y, are the same as for X-X we may write $f_s\ (aA_s + cA_s \cos \phi + dAc \cos \phi)$ for the area of steel needed to resist stresses in Y-Y.

But it was previously found that .608 sq. ins. of steel was needed to resist M in any one plane, say X-X, therefore the same amount will be needed for the equal stress in Y-Y; so

$f_s\ (aA_s + bA_s + 2cA_s \cos \phi + 2dA_s \cos \phi) = f_s\ (2 \times .608)$ for an area one foot square.

As f_s is constant and bands a, b, c, and d are equal

$$A_s = \frac{2 \times .608}{2.4 \times .707} = .252 \text{ sq. in. per ft.}$$

If the area of a ⅜-in. bar = 0.1104, the spacing of the bars equals $\frac{.1104}{.252} \times 12 = 5.25$ inch., c. to c.

Now if bands are made the width Mr. Turner recommends, i. e., $7/16L$, the number of bars in a band will equal $\frac{20 \text{ ft.} \times 12 \text{ ins.} \times 7}{16 \times 5.25} = 20$ ⅜-in. round bars per band.

If it is assumed that the stresses in the center of the slab are approximately but one-half of those at the column, but one-half the amount of steel will be needed. There actually is .252 and $\frac{.608}{2} = .304$ are needed; but as the bars are crossing in the center of the span and as tests of expanded metal seem to prove that the concrete develops higher resisting powers when reinforced with metal in the form of a mesh, this may perhaps stand as it is.

The deflection by this method, using $E_c = 3{,}000{,}000$ equals $\frac{15}{512} \frac{2.08 \times 240^4}{3{,}000{,}000 \times 7^3} = .196$ ins.,—a little more than $^3/_{16}$ ins.

The size of the column head of course will serve to modify more or less the results obtained by any method, and the writer can think of no better way than to make the diameter of the cap of such a size that the vertical shear per square inch at its perimeter will not exceed 80 lbs. per sq. in.

4. Mensch's Method.

The writer has not attempted to analyze this method but has simply copied the results given in the tables on p. 6 of Mensch's "Reinforced-Concrete Pocket Book," without demonstration.

Thickness of slab = 8 ins.
Area of steel in strip per foot of width, 0.75 sq. in.
Width of strip, 7 ft.
Size of column capital, 4 ft. 6 ins.
Weight of steel per sq. ft., 5.3 lbs.
Total weight in panel = 20 × 20 ft. × 5.3 lbs. = 2,120 lbs.
The percentage of steel per ft. width = 0.89 of 1%.

Mensch states that the rods running in the direction of the columns should extend into the adjoining panel $0.12L$, and those running diagonally should extend $0.16L$ beyond the column center.

5. Turner's Method.

Let $M = \frac{WL}{50}$ and $f_s = 13{,}000$ lbs. per sq. in.

All other quantities have the same index as in previous cases.

$L = 20$ ft.
w = Live load = 200 lbs. per sq. ft.
w^1 = Dead load = 8-in. slab = 100 lbs. per sq. ft.

1. $W = (200 + 100) \times 20^2 = 120{,}000$ lbs.
2. $M = \frac{120{,}000 \times 20 \times 12}{50} = 576{,}000$ in.-lbs.
3. $A_s = \frac{576{,}000}{.86 \times 7 \times 13{,}000} = 7.38$ sq. ins.

The area of a ⅜-in. round bar = 0.1106

4. $\frac{7.38}{.1106} = 66.7$ ⅜-in. round bars, say 68.
5. Number of bars in band $= \frac{68}{4} = 17$.

If $f_s = 16{,}000$ lbs.

$$As = \frac{576{,}000}{.86 \times 7 \times 16{,}000} = 6 \text{ sq. ins. nearly.}$$

$\frac{6}{.1106} = 54.2$ bars ⅜ in. diameter.

$\frac{54.2}{4} = 13.55$ bars, say 14 bars in each band.

6. With of band $y = \frac{7 \times 20}{16} = 9$ ft. 9 ins.

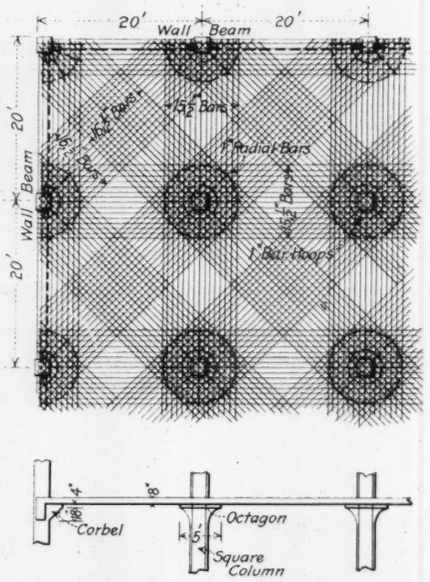


Fig. 6. Typical Diagonally Reinforced Flat-Slab Floor.
(Designed by the MacMillan method.)

Mr. Turner does not state what rule he uses to obtain the size of his column capitals.

The deflection of the slab by Turner's method

$$\Delta = \frac{1}{7000} \times \frac{(60) \times (20)^3}{(4 \times 17 \times .1106)\ (7)^2} = .19, \text{ about } ^3/_{16} \text{ in.}$$

6. MacMillan Method.

Let w = live load per sq. ft.
Let w_1 = dead load per sq. ft.
Let $w \div w_1 \times L \times B = W$ = total load on pier.
Let L and B = length and breath of panel respectively in feet.

When panel is square, $L = B$
When panel is rectangular, L is greater than B and must be used in getting the bending moment.

Let f_s = tension in steel = 16,000 lbs. per sq. in.
$2\ d$ = effective depth of slab
$2\ jd$ = arm of forces = 0.86
$2\ A_s$ = area steel = total number of bars in panel × area of single bar.
Let y = width of bands of steel = $7/16L$ approximately.
Let v = fiber stress of concrete in shear = 80 lbs.

Mushroom Floor.

Let T = total thickness of slab,

Then (1) $W = (W + W) \times L \times B$

(2) $M = \frac{W \times L \times 12}{25}$

(3) $\Sigma\ A_s = \frac{M}{jd \times f_s}$

(4) Number of bars $= \frac{\Sigma\ A_s}{\text{Area of bar used.}}$

(5) Number of bars in band $\frac{\text{total number}}{4}$

(6) Diameter of cap $= \frac{W}{\pi \times T \times V}$

(7) $y = \frac{7L}{16}$

jd will vary according to $\frac{E_s}{E_c}$ and to the percentage of steel used, but 0.86 is a good average. It will not be found advisable to make T less than 7 ins., regardless of load on account of the difficulty found in embedding the bars in concrete.

To illustrate the case we have been using,

Let $L = 20$ ft.
$W = 200$ lbs.
$w_1 = 100$ lbs. for 8-in. concrete slab.
$d = 7$ ins.
$T = 8$ ins.

1. $W = (200 + 100) \times 20 \times 20 = 120{,}000$ lbs.
2. $M = \frac{120{,}000 \times 20 \text{ ft.} \times 12 \text{ ins.}}{25} = 1{,}152{,}000$ in.-lbs.
3. $\Sigma\ A_s = \frac{1{,}152{,}000}{0.86 \times 7 \times 16{,}000} = 11.95$ sq. ins.

The area of a ½-in. round bar = .196.

4. $\frac{11.96}{.196} = 61$ bars.
5. Number of bars in band $= \frac{61}{4} = 15.25$, say 16 each way diagonally and 15 each way straight.
6. Diameter of column cap $= \frac{3.1416 \times 8 \times 80}{120{,}000} = 59.6$ ins.
7. Width of band $y = \frac{7 \times 20}{15} = 8$ ft. 9 ins.

(See Fig. 6.)

It will be desirable to get the depth of cap at the junction with column by solving for depth at that point, having shear and diameter of column fixed.

The radial bars and rings have generally been made of 1-in. round rods, the rings being lapped at the ends about 2 ft. and U-bolted together, or else welded. These rings are placed so that they bring the slab bars up near the top of the slab at the columns.

In general $f_s = 16{,}000$ has been used for stress in tension with plain round bars. The percentage of steel has been kept below 0.5% for a single layer of bars.

The wall beams are designed to carry ¼ of the panel loads and, if necessary, the columns are calculated for stresses caused by concentric loading.

The wall panels are made the same as the interior panels, and where the holes of a reasonable size occur, they have been made in the slab without hesitation.

The deflections occurring have never been calculated, but the tests made indicate that they are quite insignificant. This method has been used by the writer in connection with the following work by various engineers and architects: In designing the floors and roof for the Congress Building at Portland Me.; for a fire station at Weston, Mass.; for the floors of the East Boston Terminal Refrigerating Co.; for a storage building for the Pierce-Arrow Motor Car Co. at Buffalo, N. Y., for a storage building for the Hood Rubber Co., at Watertown, Mass.; and with a slight modification for the Winchester Repeating Arms Co.'s new loading buildings at New Haven, Conn. With the exception of the East Boston Terminal Refrigerating Co.'s floors, all of these buildings were built by the Aberthaw Construction Co.

It would seem that if the writer's interpretation of Turneaure and Maurer's diagrams and descriptions are correct, the results are extremely heavy—much more so than any existing work would lead one to believe necessary.

Only recently, in fact, the writer agreed to test a slab in the Refining Bldg. for the Hood Rubber Co. with a live load of 450 lbs. per sq. ft. This slab was 9 ins. thick and the reinforcement consisted of ½-in. round bars 7 ins. on centers. Columns were spaced 20 ft. on centers. The slab was cast 8 ins. thick, and later one inch of finish was put on. A box without a bottom whose sides were long enough to span from column center to center, and five feet high, was built, resting on the concrete slab and enclosing one bay. In the center of this bay a target was attached to the under side of the slab so that readings for deflections might be taken. The box was then filled with sand. At a load of 200 lbs. per sq. ft. no deflection of the slab could be measured. At this point, as the slab was cast in cold weather and was quite green,

was decided to put some shores under the panel and then continue the loading to the required amount. Later, when the concrete had a little more chance to season—that is, in February, 88 days after casting, the shores were taken out and the whole load was allowed to come on the slab.

The deflection under the total load was $^5/_{64}$ in. This is about $^1/_{3000}$ of the span with over twice the figured live load superimposed. This certainly does not seem to indicate that as thick a slab or as much steel as this method seems to demand was necessary. A table giving the weights of steel required by the different methods may be of interest and will be given here:

Method.	Thickness of slab, in ins.	Fiber stress per sq. in. in steel.	Lbs. steel in panel.	Deflection.
1. Cantilever	8	16,000	2,189	
2. Turneaure and Maurer.....	12	16,000	1,931	
3. Grashof	8	16,000	784	.195 in.
4. Mensch	8	16,000	2,120	
5. Turner a.	8	16,000	549	
b.	8	13,000	718	.19 in.
6. MacMillan	8	16,000	1,084	
Average (not including 5b)	8.66	16,000	1,463	

A few points worth mention are noticeable in this summary: First, the similarity of methods 1, 2, and 4 in regard to amount of steel. This holds true of methods 3 and 5b also. Secondly, the marked difference in thickness of slab required by the second method. Thirdly, the remarkable closeness in deflections obtained in 3 and 5b.

Regarding the merits or demerits of the different methods, the writer can say little except that the floors designed by his method have been low in cost, have shown remarkable powers of resisting abuse, have stood tests of twice the live load over an entire bay not only without signs of failure, but with such trivial deflection that one is led to believe that the constituent materials are far from being stressed inordinately.

It may be well to call attention to one other point that may give trouble to the designer of this system—that is the ratio of length to breadth of panel. It will be apparent that as the greatest length is used in getting the bending moment, the depth of the slab and the amount of steel increase rapidly with the increase of length and the variation of stresses will become very uncertain. In consequence, it would seem that nearly square slabs were desirable and that an increase of length over breadth of more than 25% was perhaps not economical practice.

The most economical spans would seem to be from 16 to 24 ft. For shorter spans the cost of columns runs up pretty fast. A heavy floor load seems to show greater relative economy than a light one. Where speed of erection is a desideratum, the flat floor slab with its absence of beams is far ahead of any other method.

THE MINERAL PRODUCTION OF CANADA for the year 1909, according to the preliminary report prepared by Mr. John McLeish, Chief of the Division of Mineral Resources and Statistics of the Canadian Department of Mines, had a total value of nearly $90,500,000, surpassing the value of the 1908 production by a little more than 5%. The value of the 1909 production is equally divided between metallic and non-metallic products.

The gold production decreased slightly from that of 1908 when its value was $9,842,105. Increased silver production is reported from both British Columbia and Ontario.

The copper production shows an increase estimated at about 2% over that for 1908. Refined copper is not produced in Canada, but the estimated weight exported in 1909 in the form of ores and matte or blister copper was about 54,061,000 lbs.

The production of lead was about 47,000,000 lbs., as compared with about 40,900,000 lbs. in 1908. The largest production of nickel on record is indicated although no definite estimate is made of its exact amount. The nickel production in 1908 was 19,143,111 lbs.

Total shipments of iron ore from mines in Canada in 1909 amounted to 268,043 short tons. In 1908 the production was 238,082 tons. The pig iron production shows an increase of 20% over that of 1908, and its total in 1909 was 757,162 short tons.

The coal production decreased by about 4.4% from that of 1908. The total for 1909 is estimated at 10,411,955 short tons. This is smaller than in either of the two preceding years. Nova Scotia contributed 54.5% of the total production and British Columbia contributed 24.3%.

The petroleum production, derived chiefly from the Ontario peninsula, appears to have fallen off about 20%, as indicated by bounty payments. The production in 1908 was 527,987 bbls.

The production of Portland cement amounted to about 4,090,000 bbls., an increase of 17% over the production for 1908. Very little cement is exported from Canada. The consumption of cement in the Dominion has a little more than trebled since 1905.

The total value of the mineral production in Canada in 1909 was nearly nine times that for 1886.

The Sewage Testing Station of the Sanitary District of Chicago.

By LANGDON PEARSE, Assoc. M. Am. Soc. C. E.*

For the past year much consideration has been given by the Sanitary District of Chicago to the question of the future development of the present scheme of sewage disposal by dilution. At present there are about 2,000,000 persons sewering into the Chicago River and Drainage Canal. All this sewage is swept down the canal by the induced flow from Lake Michigan. Under the charter of the District, the Sanitary District is required to use 3 cu. ft. per sec. for every 1,000 people who sewer into the canal. But the War Department at Washington has taken the stand that the Sanitary District should not be allowed to use more than 10,000 cu. ft. per sec. from the lake for the purpose of dilution. This would mean that when the population sewering into the Drainage Canal reaches the number of 3,000,000, or thereabouts, the capacity of the canal will be exhausted, as far as purification by dilution only is concerned, and that some treatment of the crude sewage would have to be used.

Disposal by dilution is a recognized method of sewage disposal. Within certain limits, it will perform its purpose satisfactorily, just as in the case of the Drainage Canal. But owing to the necessity of sufficient dissolved oxygen in the diluting water to help oxidize the incoming sewage, and to stimulate the biological action, and the time element for the action to occur, the amount of sewage which can be handled without nuisance is definitely limited. Hence, for sanitary reasons, sewage cannot be dumped in ever increasing amounts into the canal, unless more water is diverted into the canal from the lake, pro rata, in accordance with the provision of the charter. The stand of the War Department blocks at present any addition to the capacity from that direction.

The present study of the sewage disposal problem in its broad aspect has been undertaken, therefore, not because of any dissatisfaction with the present method, but to provide for the immediate future. It will furnish the Sanitary District with accurate knowledge of what the dilution method is accomplishing, as well as the adaptability of other methods, under the local conditions, and will furnish the data to determine the cheapest and best plan for increasing the present facilities for sewage development in each locality, as the population becomes more dense.

In the Calumet District, the question has arisen whether it is advisable to construct some form of sewage disposal plant and discharge the effluent into the lake, or instead to build a canal to carry the sewage into the Drainage Canal at Sag. Several reports have been made upon this problem, both by the Chief Engineer of the District, Mr. George M. Wisner, and by Messrs. Hering & Fuller. The principal reports being, "A Report to the International Waterways Commission on the Disposal of Sewage of Chicago in the Calumet Region," by Messrs. Hering & Fuller, dated, Dec. 18, 1906;* a report by Mr. Rudolph Hering to the President of the Sanitary District on the "Sewage Disposal of the Calumet Region," dated Oct. 15, 1907, and the report of the Chief Engineer, Mr. George M. Wisner, to the Board of Trustees, dated June 9, 1909.† In these reports, the matter of the disposal of the sewage by the use of intercepting sewers and a canal discharging into the Drainage Canal at Sag, has been discussed at great length, as well as the question of building and maintaining a sewage disposal plant, the effluent of which could be discharged into the lake, or into the canal, if a canal be built. The Sanitary District also includes certain isolated territory where some scheme of sewage treatment probably would be far preferable to the construction of a small canal.

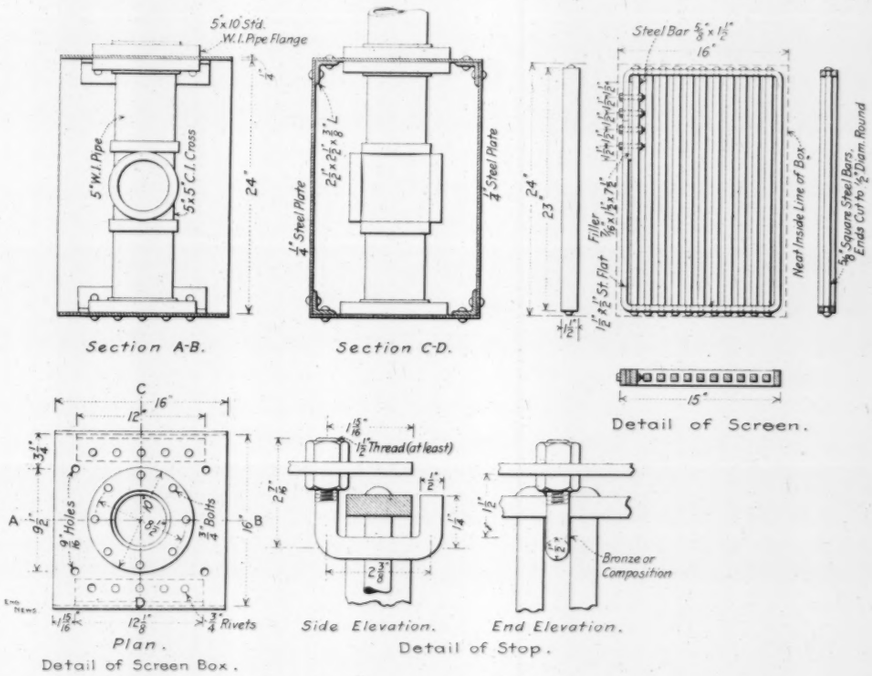


FIG. 1. DETAILS OF SCREEN BOX OR CAGE ON FOOT OF SUCTION PIPE, SEWAGE TESTING STATION, SANITARY DISTRICT OF CHICAGO.

In order to handle the problem intelligently, the Trustees of the District decided early in 1909, to make a thorough investigation of the question of sewage disposal in the territory of the Sanitary District. The first step in the investigation was to authorize the construction of a small testing station in order to study the operation of the more rapid forms of sewage treatment, such as sprinkling filters and settling tanks, particularly with reference to the climate of Chicago, as open filters of the modern sprinkling type have not been operated at this latitude. It was also considered advisable to obtain thorough data on the actual strength of the sewage throughout the District, as well as the manufacturing wastes, as this had not been observed in the past. It is proposed to study the question of screening the sewage and to determine the most economical period of sedimentation, as it is very probable that the first

*In charge of Sewage Disposal Investigations, Sanitary District of Chicago.

*Eng. News, Jan. 31 and Nov. 7, 1907.
†Engineering News, Nov. 18, 1909.

step to relieve the Drainage Canal would be the installation of screens, and the construction of settling chambers, in order to keep out of the canal a large percentage of the grit and solid matter which is at present being deposited in the channel.

At present, all the sewage of Chicago is discharged into the Drainage Canal, except that from the section south of 89th St. known as the Calumet region. The sewage from this section flows into the Calumet River along with that from many Indiana towns, and is then discharged directly into the Lake Michigan. Investigation of possible sites for the testing station showed that it was not advisable to use the sewage of the Calumet region, owing to its dilute character, and, therefore, the most available site was the one adopted, using the sewage of the residential district which discharges its sewage at the 39th St. sewage pumping station of the city of Chicago. Plans were drawn and the contracts let in the early summer of 1909. On Oct. 14, sewage was pumped into the sedimentation tanks, and by Nov. 1, the entire plant was in operation.

The sewage testing station is located on made land built out into Lake Michigan at the east end of 39th St. The greater portion of this tract is occupied by the 39th St. sewage pumping station. This situation was chosen for its accessibility, as well as the character of sewage which comes to the pumping station from a typical residential district of the city. An intercepting sewer runs from 87th St. north along the lake front which taps all the existing sewers that once discharged into Lake Michigan between 87th St. and 31st St.*

*See article by Isham Randolph, Eng. News, July 22, 1909, General Map of District.

TABLE I.—COMPARATIVE ANALYSES OF CRUDE SEWAGE OF BOSTON, COLUMBUS, WATERBURY AND CHICAGO.

(Parts per Million.)

	Boston,* 1905-'07.	Columbus,† 1904-'05.	Waterbury,‡ 1905-'06	Chicago, Nov., 1909.
Nitrogen as:				
Free ammonia	11.4	11.0	7.8	8.5
Organic nitrogen	9.1	9.0	14.8	7.3
Nitrites	0.0	0.09	0.14	0.17
Nitrates	0.04	0.20	1.52	0.18
Oxygen consumed	56+	51+	46+	35.8+
Chlorine	2,300‡	65	48	36.5
Alkalinity	125	350	41	205
Suspended matter:				
Total	135	209	165	143
Volatile	91	79	115	80
Fixed	44	130	50	63
Free CO_2	..	27	18	..

*From Winslow and Phelps. "Investigations on the Purification of Boston Sewage in Septic Tanks and Sprinkling Filters." Technology Quarterly, Vol. XX., No. 4, p. 410, Dec., 1907 (see note).

†George A. Johnson, "Report on Sewage Purification at Columbus, Ohio," pp. 26, 54.

‡Wm. Gavin Taylor, "Waterbury Sewage and Its Septic Action," Eng. News, Vol. 61, p. 597.

+Sample immersed in Boiling Water for 30 minutes.

‡Sample is boiled 5 minutes.

‡Chlorine from Water-Supply Paper No. 185 (U. S. Geol. Surv.), pp. 111-114.

Note.—Nitrogen values as given are corrected to be comparable with other figures.

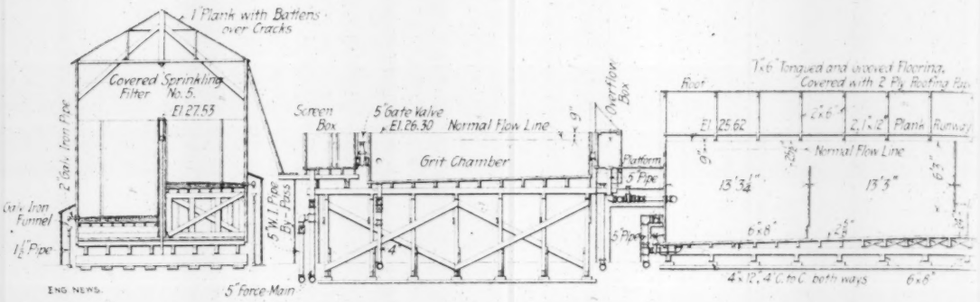
This intercepting sewer drains approximately 22.4 sq. mi. of land and receives the sewage from about 270,000 people. No exact data exist as to the amount of the water supply in the district, or the true sewage as such, but the dry weather flow of the pumping stations runs about 54,000,000 gals. per 24 hrs., which would be equivalent to about 200 gals. per capita on the drainage area. This is probably high for a residence district, but there is no direct means of correcting the estimate for the figures of infiltration of ground water. Pitometer surveys made under the direction of Mr. John Ericson, City Engineer of Chicago, show in two districts within the territory sewered, a water consumption of 247 and 163 gals. per capita daily. The sewage is dilute as compared with many cities.

During the month of November the average flow was 83,600,000 gals. daily, with a maximum flow of 293,000,000 and a minimum flow of 56,000,000 gals. The Chicago analyses given in Table I. are based on the November results only.

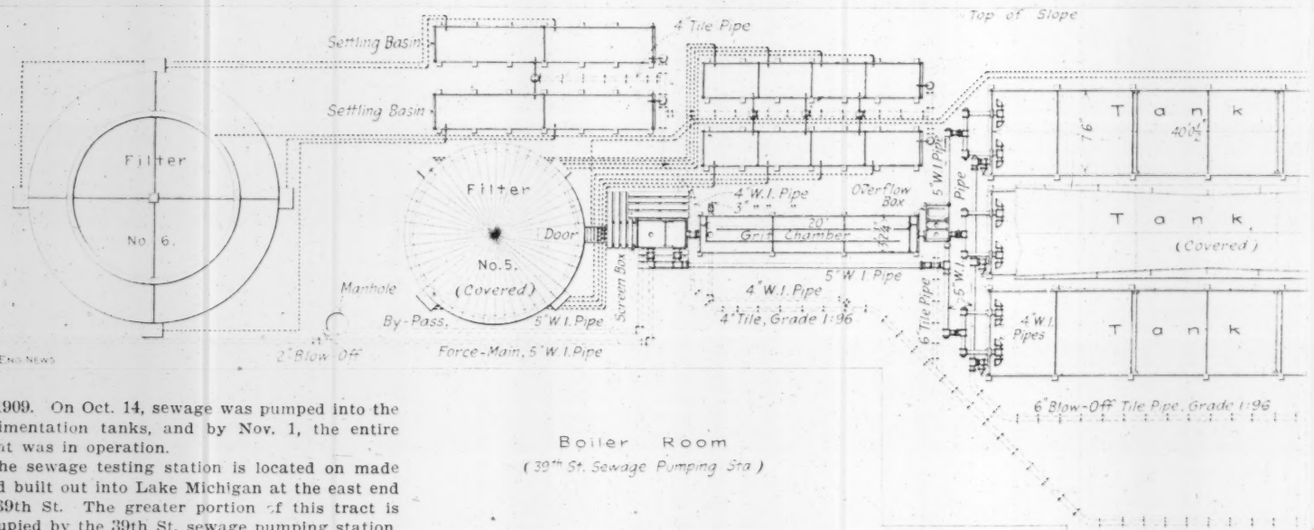
As a means of comparing the per capita strength of the sewage, Table II has been taken from the report of Mr. George A. Johnson (Sewage Purification at Columbus, Ohio), and the estimated quantities added for Chicago.

When the intercepting sewer reaches the 39th St. pumping station it is 16 ft. in diameter and discharges into a screen chamber 20 ft. wide and 30 ft. long, in which there are two coarse screens with bars set 4 ins. c. to c., which prevent chunks of wood from working into the large centrifugal pumps. On the up-stream side of these screens are located the suction pipes for the sewage testing station.

The suction pipes and force main are 5-in. wrought-



Longitudinal



Boiler Room
(39th St. Sewage Pumping Sta.)

Chimney

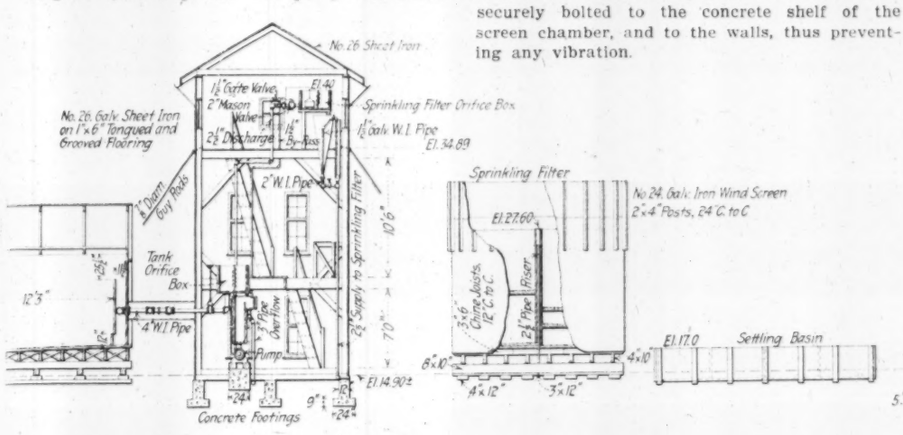
Plan.

FIG. 2. PLAN AND

TABLE II.—ESTIMATED AVERAGE QUANTITIES OF PRINCIPAL CONSTITUENTS IN GRAMS PER CAPITA DAILY OF THE SEWAGE OF VARIOUS CITIES.

City.	Columbus, Ohio. Combined.	Small Mass. cities. Separate.	London, Eng. Combined.	Manufacturing cities. Combined.	Chicago, Nov., 1909. Combined.
Combined or separate system					
Average daily flow of sewage U. S. gallons per capita	121	95	54	..	310
Oxygen consumed:					
Total	30	13	25	50	41.9
Dissolved	14	..	..	..	..
Suspended	16	..	..	..	..
Nitrogen as:					
Total	14.4	9.7	13.0	13.0	18.5
Organic:					
Total	6.2	4.7	1.5	..	8.6
Dissolved	2.4	..	..	..	..
Suspended	3.8	..	..	..	..
Free ammonia	8.2	5.0	8.0	5.5	9.9
Chlorine	32	16	24	44	43
Dissolved matters:					
Total	410	85	157	268	..
Mineral	354	58	102	178	..
Volatile	56	27	55	90	..
Suspended matters:					
Total	98	49	87	145	166
Mineral	51	11	41	69	73
Volatile	47	38	46	76	93
Total solid matters:					
Total	510	134	244	413	..
Mineral	407	69	143	247	..
Volatile	93	65	101	166	..
Free carbonic acid	13.6	..	..	..	..
Fats	19.1	..	..	..	..

iron pipes coupled up to a 2½-in. Morris horizontal centrifugal pump directly connected to a 15-HP. direct-current motor, 110 volts, running at 1,000 r. p. m. The pump will deliver 250,000 gals. per min. against 55 ft. total lift. The maximum suction lift is about 20 ft. The pumps and motor are mounted on a cast-iron base which is carried by a stiff wooden trestle, securely bolted to the concrete shelf of the screen chamber, and to the walls, thus preventing any vibration.



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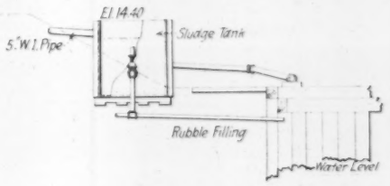
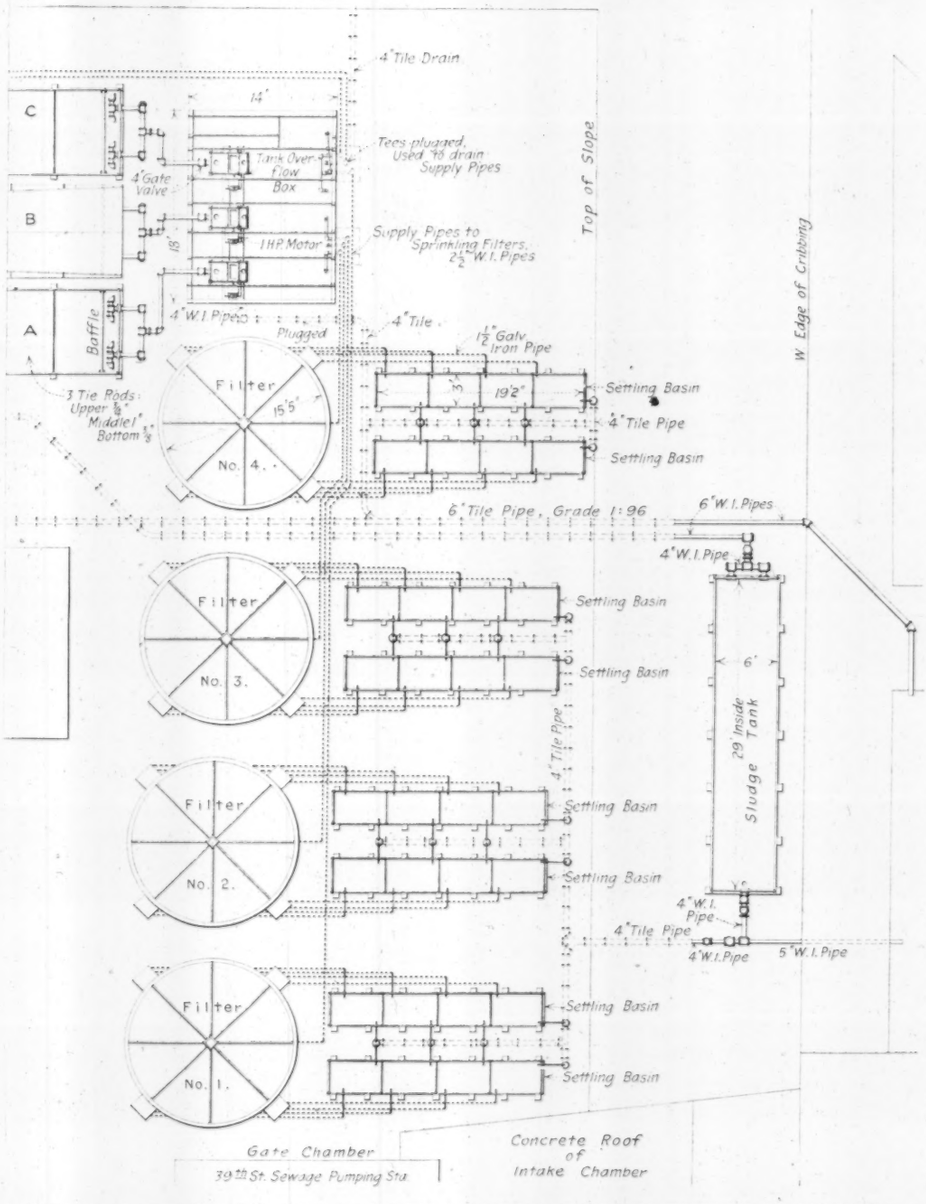




LONGITUDINAL SECTION OF SEWAGE TESTING STATION, SANITARY DISTRICT OF CHICAGO.

As the screen chamber is damp, the switch, fuses and starting box are mounted in a tight galvanized-iron box, in which two 16-c. p. incandescent lamps are always burning. This keeps the box dry. The motor is run open and develops sufficient heat to keep dry. If for any reason the pump be shut down, a tarpaulin is thrown over the motor and pump. The pump can be primed by a 1-in. pipe and a garden hose connection from the water-supply of the building, but is usually flooded back from the screen box, through the force main. The suction pipes are in duplicate, supported on iron brackets, and braced against the current. They are provided with 5-in. swing check valves set below the normal flow line.

Below the check valve is an elbow into which is loosely screwed a nipple which acts as a swivel for another elbow and pipe, to which is connected the screen box, Fig. 1. Owing to the large amount of rags, leaves and wreckage which are swept down this intercepting sewer after storms, it was considered impossible to operate an ordinary foot-valve and strainer. A special strainer box was, therefore, designed which gives about 2.6 sq. ft. of net area through the bars. The box is built of ¼-in. iron plate, stiffened at the corner by angles, and is open on two opposite sides. In these openings, which are 16 × 24 ins., are set the screens, which are built up of ⅝-in. iron bars set 1¼ ins. c. to c. These screens are held in place by bronze hooks. The screens keep out rags and coarse material, but allow leaves, paper and the like to go through.

One screen box runs downstream with the bars set vertically, and the other screen box is set at right angles to the current with the bars horizontal. Both are arranged with chains connected to the box, so that they can be held at any desired depth or lifted above the water line by swinging on the swivel elbow for cleaning. As a rule, one screen is cleaned daily and held in reserve so that when the screen in use shows signs of clogging by the diminished pumping, the suction can be shifted.

The general arrangement of the testing station is shown in Fig. 2. The 5-in. force main runs around the pumping station for about 300 ft. and discharges into a screen box in which the sewage flows through a screen. At first a brass wire diamond mesh screen was used with the wires ½-in. c. to c. The suspended matter caught on this screen was collected, weighed and analyzed for the amount of moisture and organic matter. Such a screen clogs readily and at times of storm has to be cleaned as often as every half hour. A screen with ¼-in. diameter vertical iron bars set ¾-in. c. to c. is now in use. This is cleaned with a special rake. From the screen box, the sewage passes to a grit chamber 20 ft. long, 4 ft. 3 ins. deep at one end, 3 ft. 9 ins. at the other, and 3 ft. 3 ins. wide. The bottom slope is 1 in 40. With a flow of 250,000 gals. a day and a capacity at flow depth of 1,500 gals., the velocity is about 2 ft. per min. The sewage is admitted at mid depth by means of baffles and is skimmed off at the surface with a weir at the outlet. The grit chamber is cleaned whenever the sludge shows signs of septic action. The velocities in the grit chamber are to be varied to determine the character of the sludge deposited at different rates of flow.

The sewage passes from the grit chamber to an overflow box where the excess over that required for the tanks is wasted. An excesss is always pumped. The overflow box is built with

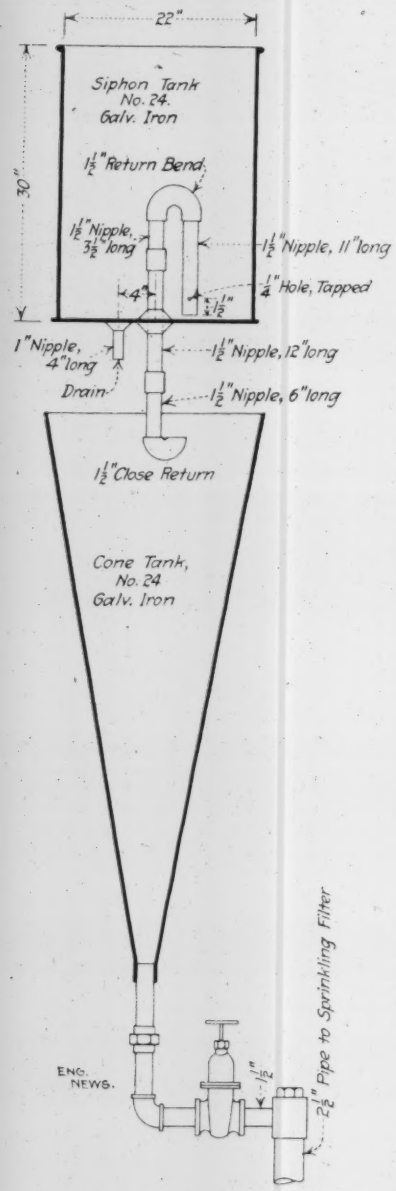


Fig. 4. Orifice Box for Sprinkling Filters, Chicago Sewage Testing Station.

two compartments, separated by a tight partition, which acts as a waste weir. One compartment is provided with a 4-in. hole which can be plugged, and with an orifice plate containing five 1⅜-in and one ¾-in. diameter standard orifices. Ordinarily the waste passes through the 4-in. hole. But once an hour this hole is plugged and the overflow is measured by the orifice plate. By maintaining a constant waste over the weir in the overflow box, a constant head is kept on the supply pipes to the tanks. This ensures uniform flow. The overflow box supplies a 5-in. wrought-iron pipe header which feeds the three large tanks by means of 4-in. wrought-iron pipe branches.

These tanks are built in standard tank construction of 3-in. yellow pine, dressed to 2⅝ ins., bolted every 18 ins. with ⅝-in. bolts through the sides and bottoms, the ends being bolted up. The chine joists are 3 ins. wide, of varied depth, spaced 2 ft. c. to c. and stiffened by bridging. The tanks are 40 ft. long inside, 9 ft. deep at one end, 8 ft. deep at the other, the bottom having a slope of 1 in 40 towards the inlet. The total capacity of each tank is 19,000 gals. As normally operated, the flow capacity is 17,000 gals., with a depth of 8 ft. at the inlet end and 7 ft. at the outlet. Two of the tanks are open, one is roofed over. Each is provided with baffles which divide the tank into three compartments. The sewage enters at mid depth through four inlets and rises to the top of the first compartment and passes down and out of the second compartment, and then is skimmed off from the surface at the outlet of the third, as shown in the section. The baffles in all three tanks are built in the same way and are made of 2-in. D. & M. yellow pine.

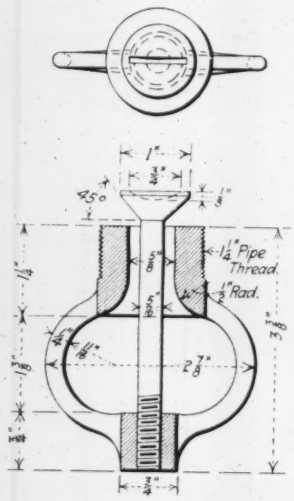


Fig. 7. Nozzle for Sprinkling Filter, Chicago Sewage Testing Station.

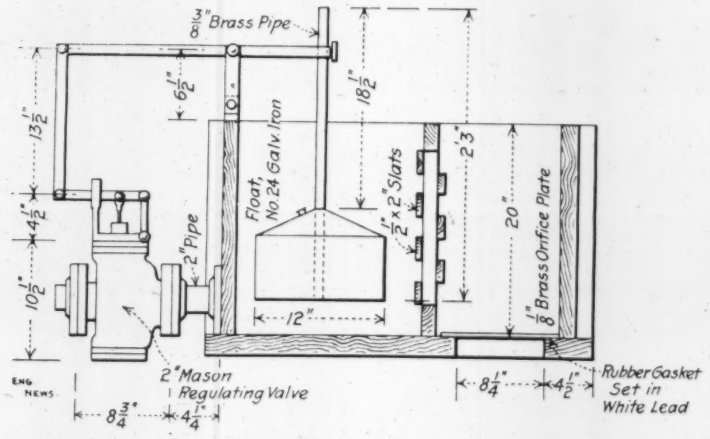


FIG. 3. TANK ORIFICE BOXES ON OUTLETS FROM SEPTIC TANKS, CHICAGO SEWAGE TESTING STATION.

(This is slightly modified from design of orifice-box published in Engineering News, May 21, 1908.)

Tank "A" is to be run as a septic tank and will probably be cleaned at intervals of six months. Tank "B" is to be run as a covered septic tank, in order to determine the difference between an open and a closed tank. Tank "C" is to be run as a sedimentation tank and will be cleaned as often as the sludge begins to show signs of septic action. The sludge from tank "C" will be discharged into a sludge tank where it will be allowed to digest with and without fresh sewage being added.

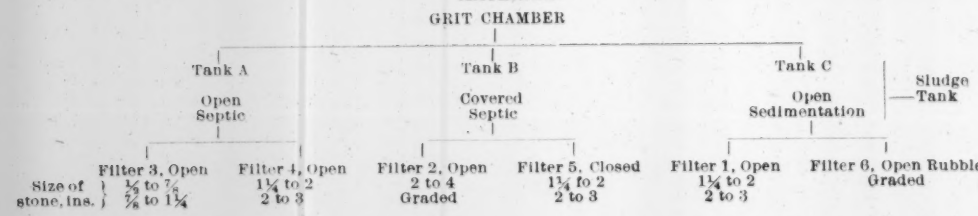
This sludge tank is 29 ft. long and 6 ft. wide and 6 ft. 10 ins. deep, with a total capacity of 8,900 gals., and is set so that the sludge flows in by gravity. The depth of flow is 5 ft. 2 ins. At present about 6,500 gals. of liquid is in the tank.

The effluents of tanks "A," "B" and "C" discharges into tank orifice boxes (Fig. 3) provided with brass orifice plates containing six ¾-in. diameter standard orifices set horizontally. The head on these orifice plates is controlled by adjusting a 4-in. gate valve on the supply line from the tank. These orifices discharge directly into overflow boxes which are connected up to the suction of 1¼-in. Morris centrifugal pumps. These boxes are all built of Oregon fir, dressed to 1¾ ins., well bolted together with ½-in. bolts.

Each pump is direct connected to a 1-HP. 110-volt D. C. motor, running at 1,275 r. p. m. These pumps will deliver 50,000 gals. per 24 hrs., against a head of 30 ft. The overflow boxes are provided with an overflow, so that the excess over the quantity pumped runs off into a drain. The tank orifice boxes, overflow boxes, pumps and sprinkling filter orifice boxes are all enclosed in a frame shed which is heated with steam coils to prevent the pipes from freezing. The discharge of each pump is a 2½-in. wrought-iron pipe, which rises to the third floor of the frame shed where each 2½-in. pipe feeds two orifice boxes. The sewage passes through a 2-in. gate valve, a 2-in. Mason regulating valve and discharges into the orifice box. These regulating valves are supplied with floats and levers to maintain a constant head on the orifices. Each sprinkling-filter orifice box is provided with a brass plate containing three ¾-in. and one ½-in. diameter standard orifices, se horizontal. The 2½-in force main is also provided with a 1½-in. valve and overflow pipe, s that the difference between the capacity of the pump and the amount used by the two orifice boxes can be wasted into the overflow box. This relieves the pumps and allows them to work at their full capacity and greatest efficiency and prevents pounding of the regulating valves.

As originally designed, each sprinkling filter orifice box discharges into a cone built of No. 24 galvanized-iron, 5 ft. deep, 2 ft. diameter at the top, and strongly soldered to a 2-in. nipple at the bottom. These cones are provided with 1½-in. siphons so that an intermit-

TABLE III.—ARRANGEMENT OF ELEMENTS OF SEWAGE TESTING PLANT AND SIZES OF FILTERING MATERIAL.



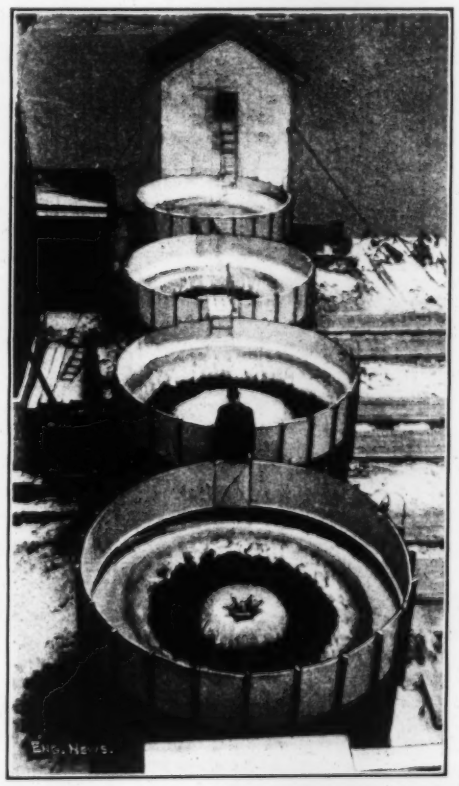


Fig. 5. Open Sprinkling Filters, Chicago Sewage Testing Station.

test and varied discharge can be secured. There is also a 1½-in. valve on the outlet from the bottom, so that the discharge may be continuous and the siphon cut out altogether, or both may be run together. This arrangement has not been found wholly satisfactory, as it builds up too high a head on the nozzles, and cannot be properly controlled. The design has therefore been changed. The cone has been lowered so that it gives a maximum head of 7 ft. for the nozzle, and a siphon tank has been installed above the cone and below the orifice box, as shown in the supplemental design (Fig. 4). This gives an intermittent discharge which can be adjusted by varying the nipple below, and will vary the head between any desired amount, usually 4 to 7 ft., according to the timing. With different rates of discharge through the orifice box, different-sized spindles have to be used in the nozzle. Each cone feeds a 2½-in. wrought-iron supply pipe to the respective sprinkling filter.

There are six sprinkling filters at present in operation. Four of these sprinkling filters are built in circular tanks approximately 15 ft. 6 ins. internal diameter with 2-in. stock Oregon fir staves, 12 ft. long. The fifth tank is 15 ft. 6 ins. inside diameter, with 16-ft. staves, 3-in. stock. This tank is covered with a light conical roof, to permit the study of the behavior of an enclosed sprinkling filter as compared with an open one.

All the open tanks are provided with wind screens of No. 26 galvanized iron, 36 ins. wide, nailed to 2 × 4-in. posts spaced 2 ft. c. to c. These make a screen to a height of about 4 ft. 3 ins. above the stone. The five tanks are each divided up into eight radial compartments, arranged in pairs of different depths, giving an effective depth for filtering material, of 4½, 6, 7½ and 9 ft. The sides and bottoms of these divisions are built of 2-in. D. & M. yellow pine. The sizes and arrangement of stone and the cross connection of the tanks with the sprinkling filters is shown by Table III.

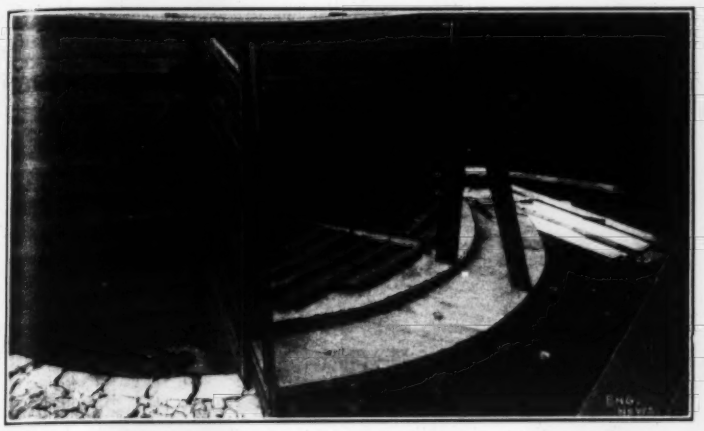


FIG. 6. UNDERDRAINS FOR SPRINKLING FILTERS, CHICAGO SEWAGE TESTING STATION.

Fig. 5 shows the completed filters (with partial ice covering); and Fig. 6 shows the underdrains.

FILTER 1.—Receives effluent from open sedimentation tank. Stone 1¼ to 2 ins. in compartments b, d, f and h. Stone 2 to 3 ins. in compartments a, c, e and g.

FILTER 2.—Receives effluent from covered septic tank. Stone 2 to 4 ins. in compartments a, c, e and g. Stone graded in compartments b, d, f and h. From top first 12 ins. is ¼ to ½-in. crushed limestone; second 12 ins. is ½ to ⅞-in.; third 12 ins. is ⅞ to 1¼-in.; all the rest, 2 to 3-in.

FILTER 3.—Receives effluent from open septic tank. Stone ⅞ to 1¼-in. in compartments a, c, e and g.; stone ½ to ⅞-in. in compartments b, d, f and h.

FILTER 4.—Receives effluent from open septic tank. Stone 1¼ to 2 ins. in compartments a, c, e and g; stone 2 to 3 ins. in compartments b, d, f and h.

FILTER 5.—Is covered and receives effluent from covered septic tank B. Stone 2 to 3 ins. in compartments b, d, f and h; stone 1¼ to 2 ins. in compartments a, c, e and g.

FILTER 6.—This filter is built with rubble walls 15 ft. 6 ins. approximate internal diameter, divided into four compartments, giving effective depths of filtering material of 4½, 6, 7½ and 9 ft. At present the filter is constructed as closely as possible after the lines laid down by Dr. Dunbar for a percolating filter with a fine surface layer. The layers are arranged as follows:

Top 16 ins. fine gravel 25 to 8 mesh per in.
4 ins. coarse gravel, 8 to 2½ meshes per in.
2 ins. ¼ to ½-in. crushed limestone.
3 ins. ½ to ⅞-in.
3 ins. ⅞ to 1¼-in.
6 ins. 1¼ to 2-in.
6 ins. 2 to 3-in.
To underdrains, 4 to 6 ins.

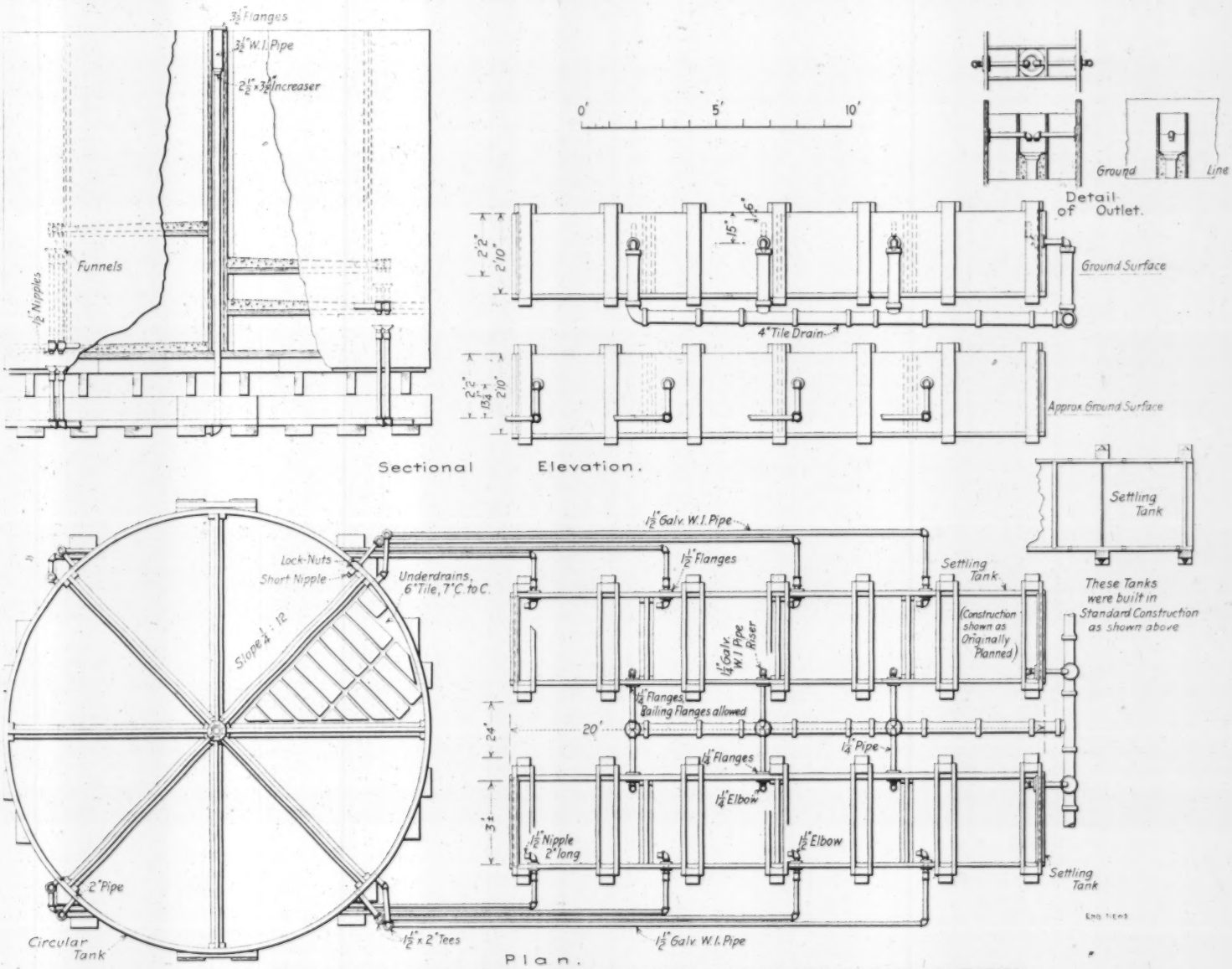


FIG. 8. PERCOLATING FILTER AND AFTER-SETTLING TANK, CHICAGO SEWAGE TESTING STATION.

The sides of this filter are built of large rubble limestone laid with open joints. At present sewage is applied to it by sprinkling, with a special nozzle of the Columbus type.

The underdrains of all the filters are composed of half tile, 6 ins. diameter, similar to the underdrains used at Columbus. The feet of the tile are embedded in a false bottom of concrete from 3 to 4 ins. thick, sloping ¼-in. per ft. Each compartment discharges into a 2-in. outlet pipe, emptying into funnels connected up with 1½-in. wrought-iron pipes, and leading to the settling basins. Ventilation is provided by an open tee on the discharge pipe, and also by an 1¼-in. hole bored into the underdrain at the point furthest from the outlet. Over the tile is laid crushed limestone, 2 to 4 ins. in size, to a depth of 10 ins. over the concrete floor at the shallowest point. This was laid level.

The stone used in all the tanks is a crushed limestone from quarries at Hawthorne and Gary. The ratings given above are the commercial sizes. For the two fine layers on the Dunbar filter, washed Lake Michigan gravel was used. The limestone as supplied contained more or less dust, as none of the stone at present on the Chicago market is washed, and, therefore, had to be washed by hand with a hose before being placed in the tanks.

Each circular sprinkling filter is provided with two settling basins, 19 ft. 2 ins. long, 3 ft. wide, 2 ft. 10 ins. deep, which contain four compartments so arranged that the division and time of settling can be regulated to suit the rate of filtration. At present two divisions or one-quarter of each filter is discharged into one compartment. The Dunbar filter is connected up with two settling basins each containing two compartments. Owing to the small amount of sewage flowing, it may not be possible to operate in extremely cold weather without covering the settling basins. The whole matter of the effect of the weather on the sprinkling filters is to be studied very thoroughly here, in order to determine whether it is necessary in the Chicago climate to cover the devices. Sprinkling filter effluent and waste sewage are all discharged into the intake of the screw pumps of the main sewage pumping station and are pumped into the Drainage Canal with the diluting lake water.

The nozzles used in the sprinkling filters (Fig. 7) are designed after the type used at Reading, Pa., being the Columbus nozzles with the arms submerged. Various-sized cones will be used to suit the different rates. Owing to the low rates of discharge necessary at times in experimental work it is not expected that the nozzles will be absolutely free from clogging. It is intended to experiment with the various types of nozzles, such as the Columbus, Weand and Taylor, and to devise a nozzle satisfactory for a large plant, to work under a variable head and also a nozzle to work under a constant head.

For protection against frost, all the exposed pipes are surrounded by light boxes which are packed with manure. The discharge pipes and funnels of the sprinkling filters are also boxed in. Ashes have been banked around the bottoms of the sprinkling filter tanks to protect them as much as possible from the cold. Fig. 8 shows one of the filters.

The plant in general is designed to be as flexible as possible. The various devices can be cross-connected by simple changes in the piping which is always accessible. The screen chamber is provided with a by-pass so that either crude or screened sewage can be by-passed around the grit chamber when the grit chamber is being cleaned. Crude sewage can also be used in tank "A" if desired, while tanks "B" and "C" are working with grit chamber sewage.

The object of the experimental station is to determine the most economical period of sedimentation for Chicago sewage, and the effect of removing the sludge from the settling tank at frequent intervals and allowing it to digest separately, and also to decide upon the most efficient sizes and depths of stone for the operation of the sprinkling filters. It is expected later on, as far as the space available will permit, that some additional devices will be installed to experiment with the dilution and sterilization of sprinkling-filter effluents, and also to study different forms of settling tanks, as well as to secure hydraulic data on the effective period of the settling of sewage at velocities of 1 ft. per min. and upward. It is also probable that other sprinkling filter materials will be tried later on, such as slag.

The laboratory for the analytical work in connection with the experimental station is located in the pumping station proper and is very compact. It is, however, efficiently arranged for the work in hand. There is a bacteriological room with a small preparation room adjacent, and a chemical laboratory with a small wash room and hood. All the available space is utilized. The office of the station is on the gallery of the pumping station.

All the construction in the testing station is of wood, as far as possible, because of its temporary construction. When the experiments are concluded, it will have to be dismantled in order to leave a suitable lawn around the pumping plant. As the station is built on made land, it was considered best not to bury the tanks in any way, but to work on the surface in order to lessen the possibility of uneven settling.

The testing station was designed and built by the writer, who had charge of the construction work, and who is also in charge of its operation. Dr. Arthur Lederer is Chief Chemist and Bacteriologist; Mr. Harry B. Hommon, Assistant Chemist; and Mr. T. H. Glenn, Assistant Bacteriologist. Mr. Geo. M. Wisner, M. Am. Soc. C. E., is the Chief Engineer of the Sanitary District, and Mr. Robert R. McCormick the President.

Further Tests on the Effect of Electrolysis on Concrete.*

By O. L. ELTINGE.†

The following experiments were conducted in the electrical and testing laboratories of the Rensselaer Polytechnic Institute, during the spring of 1909, to supplement the published data on the effect of electricity on concrete. The writer was assisted by Mr. Harold S. Beers, C. E., Instructor in the Department of Physics and Electricity. The tests were not run at the usual 7, 14 and 28-day intervals because the testing was of necessity secondary to the regular school work.

The first tests were made under a 110-volt pressure on five concrete blocks, as shown in Fig. 1, and called here, respectively, Blocks No. 1 to 5. These blocks were made of a 1:2:4 mixture by volume of a standard Portland cement, a rather dirty bank sand and crushed limestone, under ¾-in. size, with the dust screened out, and an amount of water varying from 10 to 20%, as follows:

No. 1: 13%—flushed under long tamping.
No. 2: 10%—too dry; used to get porous block.
No. 3: 15%—placed by strong spading.
No. 4: 19%—placed with very slight spading.
No. 5: 20.6%—too wet; stone and mortar began to separate.

These blocks were kept moist in forms for six days and set in 3% brine (see Fig. 1) and current passed only long enough to read the ammeter and voltmeter and resistances computed as 93, 35, 75, 61 and 66 ohms, respectively, for Nos. 1 to 5. After setting in this solution for two days the back, E. M. F. from rod (+) to pail (—) was 0.8 volts in each block. As this is so small, it can be neglected in computing the resistances. This test was run on a voltage of from 121 to 126 volts. When the blocks were 8 days old the circuit was closed and the computed resistances found to be 47, 36, 53, 37 and 37 ohms, respectively. The resistances fell off slightly for a few minutes to a minimum of 41, 29, 42, 35 and 35 ohms, respectively, and then began a gradual rise. The accompanying curve (Fig. 2) is typical of all the blocks except that the rise and fall of the curve is a little more marked in this one than in the others. Seven minutes after the test was started water was bubbling out around the rods on blocks Nos. 2, 3 and 4 and out of the pores in the sides of Nos. 1 and 2. At nine minutes steam was coming out of the pores on the sides of No. 2. At 15 minutes a continuous bubbling around the rods appeared on No. 4. At 20 minutes No. 2 was uncomfortably hot to the touch and iron rust stain showed on the suds like foam which was coming out of the sides. At 25 minutes iron rust stains showed on the foam about the rods on Nos. 4 and 5. At 35 minutes cracks showed near the corner of No. 2. The giving off of chlorine and steam, which began at about ten minutes on all the blocks, was less after one hour, but all the blocks were very hot to the touch. At two hours a thermometer laid against the surface of each block gave readings of 70°, 91°, 70°, 76° and 74° C., respectively. The thermometer was placed in the fumes that were coming from near the rod on block No. 2, so giving the high reading shown. At 3½ hours the giving off of steam and chlorine was nearly stopped and then only by intervals, but the blocks were very well dried out and hot so the test was stopped at this time with the resistances of the blocks 76, 101, 67, 85 and 69 ohms, respectively. Only one of the blocks was cracked at this time and that one (No. 2) not badly.

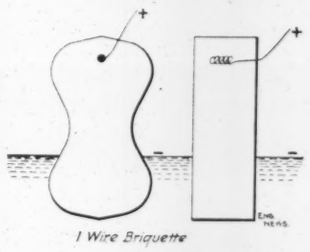


Fig. 1. Test Blocks in Concrete Electrolysis Experiments.

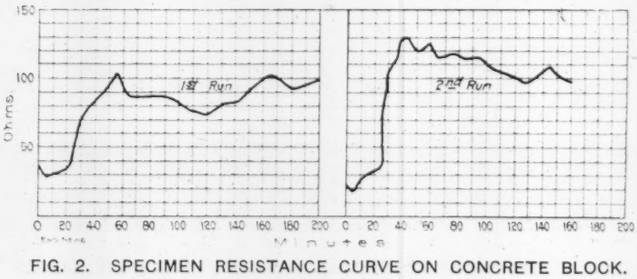


FIG. 2. SPECIMEN RESISTANCE CURVE ON CONCRETE BLOCK.
(Resistance of Block No. 2 as a function of Time.)

When these blocks were 11 days old the test was resumed with a resistance at start of 40, 23, 42, 27, and 16½ ohms, respectively. The resistances fell in five minutes to 31, 18, 32, 23 and 19 ohms, respectively. A green scum and a dark brown liquid came out of the pores of the sides of the blocks a few minutes after the test was started. The bubbling around the rods was stronger than before, but stopped sooner. At 55 minutes a crack showed on the top and side of block No. 1. At 95 minutes a crack showed in No. 4. At 100 minutes a crack showed in No. 5. At 2¾ hours the giving off of steam and chlorine again decreased and no evidence of any possible further damage showed, so the test was stopped with the resistances of 84, 97, 73, 76 and 62 ohms, respectively. In this test the steam and foam came out of new places in the sides of the blocks and not from the ones stained by the first test.

To open the blocks, a cold chisel was worked into the cracks already started in Nos. 4, 5 and 1 and the block pried open. On Nos. 2 and 3, which were uncracked, the crack was made flatways

*For previous articles on this subject see Engineering News: 1907, i, 328; 1908, ii, 710, 718; 1909, i, 458.

†905 Tracy Ave., Kansas City, Mo.

TABLE I.

Composition Treatment.	Neat. No. of test pieces.	Neat. Average breaking strength, lbs. per sq. in.	Neat. Per cent. of dummy.	1 to 1 with sand by weight. No. of test pieces.	1 to 1 with sand by weight. Average breaking strength.	1 to 1 with sand by weight. Per cent. of dummy.	20% iron turnings. 80% cement. by weight. No. of test pieces.	20% iron turnings. 80% cement. by weight. Average breaking strength.	20% iron turnings. 80% cement. by weight. Per cent. of dummy.	10% salt. 90% cement. by weight. No. of test pieces.	10% salt. 90% cement. by weight. Average breaking strength.	10% salt. 90% cement. by weight. Per cent. of dummy.	10% salt. 20% iron turnings. 70% cement. by weight. No. of test pieces.	10% salt. 20% iron turnings. 70% cement. by weight. Average breaking strength.	10% salt. 20% iron turnings. 70% cement. by weight. Per cent. of dummy.
Dummies	6	374	..	7	466	..	5	416	..	6	317	..	5	250	..
Heated dry	5	274	73	6	291	62	5	303	73	5	314	99	5	250	100
Heated in brine	5	207	55	7	281	60	5	228	55	5	302	95	6	237	95
One wire briquettes treated by electricity, one end in brine	4	309	83	7	486	105	3	228	55	6	317	100	6	180	72
Two wire briquettes treated by electricity dry	5	339	91	5	425	91	6	389	94	3	189	60	7	209	84

of the rod with hammer and cold chisel. All the specimens showed no cracks below the water line, but when cracked open the cracks would follow down. At the bars a very heavy corrosion stain showed on the concrete, stronger near the top of the block also stronger in the more porous specimens. The rods were badly eaten, the rust being caked on them. Most of the rusting was like ordinary iron rust, but was streaked with yellow salts near the edges in some places and in center close to the rod, where the action had been strongest there was much granular black salt which looked like magnetite.

In the inside parts of the blocks the concrete was very much weaker than near the surface, having the appearance of being partially crushed.

TEMPERATURE TESTS.—This test gave so much evidence of internal heat that two tests were made to determine this temperature to a better extent. In these blocks ½-in. gas pipes capped on one end were used in place of rods. When the tests were run the gas pipes were filled with machine oil and a thermometer let down into the oil till practically covered. One block which was 28 days old and had been stored in a dry place was set in 3% brine for 1½ hours and then connected to the 500 volt circuit. The temperature at the start was about 20° C., but when the circuit was closed the temperature rose so rapidly that in less than five minutes the temperature was 100° C., and then gradually increased to 125° C in seven hours. After testing this block was found to be cracked through the cap on the pipe in a plane across the layers and roughly parallel to the square base. This block took 18 amps., for part of a minute at the start but dropped in a short time to 0.4 amps. and then slowly to 0.25 amps. and then irregularly between 0.4 amps. and 0.25 amps. to the end of the test.

The other block was tested at the age of 21 days and had been stored in a damp place since making. This was tested on the 110-volt circuit. The temperature started at 20° C. and was run up to 70° C. in one hour and to 92° in four hours, where it was increasing so slowly that the test was stopped. The amperage ran up very slowly from 1.1 amps. at start to 1.7 amps. at two hours and then slowly decreased to 1.45 amps. at the end of the test at four hours.

If any error was made in reading the thermometer in these two tests for the internal temperature of the briquettes, it was an error of reading the temperature too low, as the thermometer had to be read when the mercury column was falling very rapidly in the tube, due to the lifting of the thermometer from the hot oil into the cooler air to be read. In the second test it cannot be surely told whether if the test had been continued for several hours more the temperature would have exceeded 100° C. or not, but by comparing the curves of the two tests, I would judge that it would after several hours, but not very much.

FRESH WATER TEST.—To eliminate any effect due to the salt in the electrolyte used a test was made using fresh water for the electrolyte. This block was of 1:2:4 mixture and 93 days old. It had been stored in a dry place for over four weeks, then set in water for 1½ hours before the test was started. As was to be expected the resistance was higher than with a brine for the electrolyte. Tested at 500 volts it carried a current of from 0.5 to 0.2 amp. and

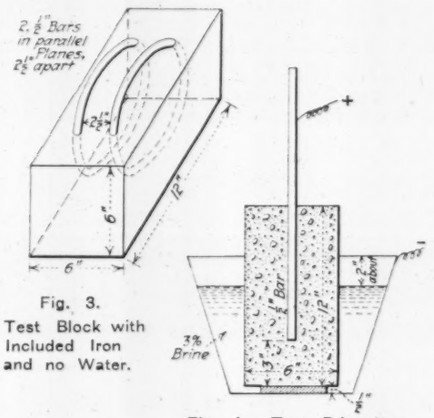


Fig. 3. Test Block with Included Iron and no Water.

Fig. 4. Test Briquette in Concrete Electrolysis Experiments.

In second test wires were connected to both ends of the briquette and current passed through without the interposition of water. (See Tables I-II.)

showed no apparent injury on the surface after 8 hours test. Two months later after storing in a wet place the briquette was broken open. The concrete was found to be quite strong and the rod slightly rusted.

DRY TEST.—To eliminate the effect of the water as much as possible a block was made with two rods bent into a circular form and placed as shown in Fig. 3. This block was tested on the 110-volt circuit at an age of 7 days for over two hours, with only the effect of drying out and heating the block strongly. Then the block was stored dry until 44 days old when it was tested on the 500-volt circuit for 8 hours. The current ran from 0.5 amp. at start to 0.7 amp. at five minutes and then fell off to 0.1 amp. at one hour and 0.025 amp. at six hours, where it stayed for the next two hours. This gives the resistance across the slab 2½ ins. thick between the two rods, of 20,000 ohms. When cracked open two months later, after storing wet, the following conditions were found: At the negative rod the concrete was strong and hard and the rod was cleaner than when the block was made. At the positive rod the concrete was weaker and broke open along the plane of the rod quite easily and the rod was rusted but not badly.

HEAT TESTS.—Two tests were now made to see if heat alone would produce cracks the same as the electricity had done. One block was made with a ½-in. bar similar to those tested by electricity. This block, at the age of 21 days, was placed so that the flame from a Bunsen burner would strike the rod 2 ins. from the concrete. In 15 minutes a crack showed. At two hours this crack was no larger, and no others had formed. The piece that had cracked loose was taken off and the concrete near the rod was found to be softer than the outside of the block the same as in the electrical tests. Later this piece was tested by putting a gas blowpipe on the rod till the rod was soft enough to bend by hand without producing any more cracks in the concrete. In the other test a 3-in. gas pipe was set in the briquette in place of the rod so as to get more expansive force with a given rise in temperature of the rod. The pipe was heated near the concrete with a Bunsen burner, but as the heat was radiated by the large pipe surface as fast as taken up no results were found. Then a gasoline blow torch was turned onto the pipe near the concrete for 20 minutes, producing three cracks. In handling this briquette immediately after the test it was found that there was water in the back end of the pipe which had not been heated enough to show as steam.

BRIQUETTE TESTS.—To see if the apparent softening of the concrete near the rods was a rotting due to the electricity, the following series of tests were made on cement briquettes. To cause the current to flow through the neck of the briquettes the method shown by Fig. 4 was used. The following mixtures were used: (1) neat; (2) 1 to 1 with a coarse bank sand; (3) 20% cast-iron turnings of the same size as the sand and 80% cement; (4) 10% salt and 90% cement to see if the salt had any effect on the strength; (5) 20% iron turnings, 10% salt and 70% cement. All the mixtures were by weight and the briquettes containing salt were stored in a concentrated brine till tested in order that the salt should not be absorbed out of the briquettes, so weakening them. These briquettes were treated at the age of 21 to 25 days with a current of 110 volts for seven hours. During the first hour of the test 25% of the briquettes were cracked by the flow

TABLE II.

Composition Treatment.	Neat. Average breaking strength lbs. per sq. in.	Neat. % of dummy.	50% sand. 50% cement. By weight. Average breaking strength.	50% sand. 50% cement. By weight. % of dummy.	25% cement. 75% sand. By weight. Average breaking strength.	25% cement. 75% sand. By weight. % of dummy.	20% iron turnings. 80% cement. By weight. Average breaking strength.	20% iron turnings. 80% cement. By weight. % of dummy.	20% iron turnings. 30% sand. 50% cement. By weight. Average breaking strength.	20% iron turnings. 30% sand. 50% cement. By weight. % of dummy.
Dummies	726	..	659	..	386	..	731	..	651	..
Heated Dry	335	46	349	53	244	63	367	50	402	62
Heated in Brine	406	56	488	74	369	96	581	80	524	81
One wire briquettes treated by electricity with one end in brine	532	73	540	82	359	93	609	83	567	87
Two wire briquettes treated by electricity dry	497	68	527	80	389	96	526	72	616	95

(7 test pieces were used for each average noted above.)

TABLE III.

	Dummies.	Dummies.*	Treated; no rest.	Treated; no rest*	Treated; 3 days' rest.	Treated; 6 days' rest.		
Breaking strength	600	...	675	...	705	725	600	775
	680	...	440	...	705	715	640	705
	795	...	430	...	685	675	715	785
	660	...	445	...	605	730	740	610
	690	...	645	...	650	730	615	645
	780	...	3 briquettes	...	615	770	630	555
	600	...	lost due to	...	695	720	650	730
	855	...	accident.	...	725	500	675	775
Average	708	726	527	497	673	696	658	698
Average	717		512		673	684		
Per cent. of dummies	...	...	72%		94%	95.5%		

*From Table No. 2.

of current, and of these one-half were of the mixture 10% salt and 90% cement. All the cracking and steaming occurred during the first three hours of the test, after which the briquettes were heated and dried out only. At the same age part of the briquettes were heated dry in an oven to a temperature of 145° C. for 3½ hours. Another part was boiled in a saturated brine for 3½ hours at a temperature of 109° C. After the treatment in these different ways the briquettes were put back into water or brine and left till they were broken at the age of 28 days. The results are given in Table I.

The results are erratic, due to the mixture being too dry and difficulty in placing the wires. But the results are given as a check on the next set of experiments shown in Table II. In this second set soft iron wire was used in place of the copper wires used in the first set. The briquettes that were treated by electricity were exposed to the current at 50 volts for 15 minutes, before the 110-volt current was turned on in order that they be dried out and so escape the loss of briquettes due to cracking. The briquettes with one wire were connected to a rheostat so that the current through them this time would be equal to that through the briquettes with two wires, differing from the other test in that in the first set the voltage across both groups of briquettes was equal. This treatment lasted for four hours or until all apparent damage had been done that the current could do. The briquettes simply heated were heated at a temperature near 100° C. for two hours. These briquettes were from 36 to 50 days old when broken. They were broken immediately after treatment in order that they would have no chance to recover from the treatment. The results are given in Table II.

To see if the cement would recover from the shown loss of strength a test was made of neat cement briquettes broken at an age of 28 days. Those allowed to rest were stored in water for that length of time. They were of the two-wire kind and were treated the same as those of the same kind in Table II. The results are shown in Table III.

In a test started but not finished for lack of time the heating and drying out effect of the current and, to a slight extent, the bubbling of the water at the electrodes were just noticeable at a voltage of 4.3 volts and a current of 0.05 amp. on six small cubes of neat cement with two wires embedded in each of them, connected in parallel. And on a similar test of 16 two-wire cement briquettes, connected in parallel, the same effects were noted at an E. M. F. of 4.3 volts and a current of 0.02 amp.

Conclusions.

From all of the above I would conclude that the direct damage of electricity is practically nothing, but that the heating induced by the current does damage by drying out the concrete, so reducing its strength and also if allowed to continue for some time it will produce dangerous cracks by temperature stresses. Concrete itself is a very poor conductor of electricity, but will carry a large current by virtue of the absorbed water held in the pores of the concrete. Any electrolytic action that would be caused by having the water or brine as an electrolyte will go on and not affect the concrete appreciably except by the heating caused. I would account for the cracking of the concrete in the following way: The flow of current will produce heat most strongly near the rod, and as concrete is a very poor conductor of heat, this heat will not be conducted away as fast as produced, and so the temperature of the rod will be greater than that of the adjoining concrete, and this concrete in turn will be hotter than the outside of the block. This heat, causing the rod to expand, will put the adjoining concrete in compression, and this compression will cause the outside of the specimen to be in tension. Now the heating of the concrete near the rod will cause this to expand also, so putting it more strongly in compression and the outside more strongly in tension. When these temperature stresses become large enough they will cause the block to crack open. Then any deposit of oxides or chlorides near the rod will tend to enlarge these cracks.

Results of Tree Planting by Railways.*

(1) No ties have yet been produced from trees planted by any railway.

(2) Of the total number of trees planted, the locusts predominate, with catalpa second. Results to date favor the former, though it is perhaps too early to fairly estimate the ultimate value of any of the plantations now under cultivation.

(3) With the exception of 1907 and 1908 there has been, since 1900, a steady increase in the number of trees planted; indicating that more attention is being given the question of providing a future tie supply.

(4) The railway which has planted the greatest number of trees has decided that tree planting is the best way to utilize its idle outlying lands.

(5) While a large percentage of roads reporting plantations also report failures (or at least unsatisfactory present results), the information received indicates that the failures, to a large extent, are due to unfair experiments. Where plantations are contemplated it is recommended: (A) That the species of trees to be grown be selected only after careful and expert study of the available soils and climate; and (B) That where plantations are made, they be of such extent as to warrant a maintenance appropriation sufficient to insure proper attention.

(6) Tree culture should be encouraged where practicable. The number and variety of experiments should be increased each year, to the end that the trees most suitable for the various climates, soils, etc., may soon be known and their production specialized. This is important, because, with the timber supply rapidly diminishing, it is unwise to plant any one species and then defer other planting for five or ten years awaiting results of the first planting, which may be a failure.

(7) Where railways have outlying lands upon which second-growth timber is appearing, or where a natural original forest is growing, those tracts should be carefully preserved and the native timbers added to, if practicable; especially if those tracts contain trees of the semi-hard varieties, which can be effectively treated, and are of comparatively rapid growth.

Chicago, Burlington & Quincy Ry.—No definite knowledge. Conditions are favorable for obtaining, within about 20 years, ties and posts amounting to double the expenditure and interest on property up to the present time. Expect to plant 10,000 trees in 1910.

Pennsylvania Ry.—Too early to estimate returns, but are certain forest planting is best way to utilize company's outlying idle lands.

Delaware, Lackawanna & Western Ry.—Lost about 3,000 trees planted on low ground; did not stand the winter. Balance are thrifty and look well.

Galveston, Harrisburg & San Antonio Ry.—No reliable estimate of ultimate value can be made. Present results do not warrant additional planting. Catalpa is decided failure in arid and semi-arid districts in Texas. In the humid districts of Louisiana and Texas conditions do not seem favorable to its growth.

Southern Ry.—Too early for any estimate as to ultimate value. Trees have secured fair root growth and it is expected to cut all crooked and branched trees to ground line this winter. About 5,000 seedlings failed to take root at time of planting.

Michigan Central Ry.—Total of 123 acres planted. Latitude of extreme coldness was unsuitable for catalpa, and practically none of the trees grew.

Norfolk & Western Ry.—Soil and climate found unsuitable and experiment discontinued. Company has large holdings of mountain timber, also tracts in eastern Virginia, containing both soft and hard woods, original and second growths, which are being held; only mature timbers being cut.

St. Louis & San Francisco Ry.—Of 640 acres planted, only about 250 acres bore trees; balance were on poor, thin soil. Trees would never make ties because of poor soil, so were used for posts and poles.

Southern Pacific Ry.—Expects to soon start planting sugar and red gum.

Union Pacific Ry.—Contemplates planting catalpa on low lands owned by company.

Delaware & Hudson Co.—Owns large tracts in Adirondacks and is now making experiments to determine what trees are best suited to soils and climate.

Northern Pacific Ry.—Has reserved from sale several million acres of its timber lands; products from these tracts to be used for future tie supply.

C., C., C. & St. L. Ry.—Started small plantation near Indianapolis some years ago; land used was not favorable and trees were not of proper species. Never obtained any results.

*Appendix to report of Committee on Ties, presented at the annual meeting of the American Railway Engineering and Maintenance of Way Association, held at Chicago, March 15 to 17.

PASSENGER TRAIN MOVEMENTS at the Grand Central Station, New York City, in 1909, are summarized as follows in a statement issued by the New York Central R. R. Notwithstanding the fact that the work of construction and excavation for the new station and terminal yards was in progress during the year, this station handled 177,802 trains in the year or an average of a train every three minutes during the whole 24 hours. The aggregate length of these trains would be 14,143 miles. The total delay in handling at the terminals amounted to 36,563 minutes or an average detention of only 12 seconds per train.

TREE PLANTING ON AMERICAN RAILWAYS.

Railways.	Locations.	Trees: Date.	Trees: Number.	Trees: Kind.	Cost per 1,000: First cost.	Cost per 1,000: Planting.	Cost per 1,000: Annual maintenance.
Chi., B. & Q.	Ottumwa, Ia.	1906	100,000	Catalpa	$10.00	$20.00	No figures available
Pennsylvania	40 plantations between New Brunswick, N. J., and Altoona, Pa.	Some each year 1902 to 1909.	2,000,000 1,200,000 230,000	Black locust. Red oak. Mixed species.	$5.00 to $10 from commercial nursery. $3 to $5 now from company nursery.	About $5.00 at present.	Practically no charges for maintenance.
		Total	3,430,000				
L. & N.	Eleven locations.		868,930 570,000 630,800 14,650	Catalpa Walnuts. Locust. Poplar.	$5.64	$23.44, includes maintenance charges to date.	.....
		Total	2,084,380				
D., L. & W.	Towaco, N. J., and Alden, N. Y.	1906 1907	44,000 50,000	Yellow locust. Yellow locust.	$23.02	$12.22	$1.20
		Total	94,000				
Galv., Har. & San Antonio..	At section houses between New Orleans and El Paso.	1903	15,000	Catalpa	$16.70	$12.00	$50.00
Southern	Wolf Trap, Va.	1906	40,000	Catalpa	$19.40	$7.77	$2.84
Michigan Central	Small plantations at section houses.	1900 1903	80,000	Catalpa	No figures given	No figures given	No figures given
Norfolk & Western	Ivor, Va.	1905	2,000	Catalpa	No figures given	No figures given	No figures given
Penna. (N. W. Sys.)	Kosciusko, Ind.	1906	35,000	Catalpa	$12.65	$10.46	$16.85
S. A. & A. P.	Skidmore, Tex.	1909	22,000	Catalpa	$11.60	$27.10	$32.72
St. L. & S. F.	Farlington, Kan.	1880	No figures given		.....	.....	.....

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The printed testimony taken by the Congressional Committee which is investigating the conduct of the Interior Department by Secretary Ballinger has now accumulated to over 2,000 pages. Most of the testimony has related to the coal lands in Alaska, and the efforts made to prevent their acquirement by the Guggenheim-Morgan interests.

Some of the testimony, however, has covered the action of the Secretary toward the Reclamation Service and we have deemed it well worth while to print in this issue some extracts from the testimony of Mr. Arthur P. Davis, Chief Engineer of the Service. We commend this testimony to the careful perusal of our readers and we believe that all who know the circumstances will agree that the engineering profession owes a large debt to Mr. Davis for his frank and outspoken defence of his associates.

It is apparent to those who can read between the lines that it is only due to Mr. Davis's courage and tact and loyalty to duty, that Secretary Ballinger has not made far worse havoc with the Reclamation Service.

The Secretary assumed office with a strong prejudice against the Reclamation Service and all its works, a prejudice that reveals his sad lack of the judicial spirit notwithstanding his title of Judge. It would have been easy for a weaker man and a less loyal man in Mr. Davis's place to have yielded to, and humored, that prejudice to a certain extent, and thus to have curried favor for himself at the sacrifice of his associates and his Chief. That was the easy way. The hard and difficult way was to stand up manfully in defence of the Service and the men who had built it up and to do it with such tact and discretion as to retain his influence with and hold upon the Secretary.

We regret that space does not permit the printing of fuller extracts from Mr. Davis's testimony. The position he was placed in as a witness was most trying. It should be said that he was summoned to the stand and did not appear as a voluntary witness. Had he volunteered his evidence, he might, of course, have been accused of disloyalty toward the Secretary, his superior. Once upon the stand, however, he testified to the whole truth without fear, favor or prejudice. He made a strong defence of the Reclamation Service, and did not mince words in his statements as to the results of Mr. Ballinger's administration; but he was careful at the same time to do no injustice to the Secretary and not to attack his motives.

There is a sentence in Mr. Davis's testimony which is so significant that it deserves to be quoted in capitals.

THE HIGHEST FUNCTION OF ADMINISTRATIVE ABILITY IS THAT OF RECOGNIZING ABILITY IN OTHERS.

It is through the carrying out of that principle that the Reclamation Service has been built up. The disregard of that principle can tear it down. We sincerely hope that for the remainder of the present administration constructive and not destructive principles and methods may be permitted to prevail in the Reclamation Service.

When such great natural advantages as admirable water-transportation facilities and steep hillsides pressing upon a water-front are found combined in a rapidly growing city, it may become necessary to enter upon extensive regrading operations. Work of this character, which created some comment at the time, was done years ago at Omaha, and we believe at St. Louis and Kansas City. But the regrading at these points was of little account compared with the vast operations which have been going on at Seattle for a number of years past. Elsewhere in this issue, we supplement our earlier articles on the Seattle regrading work by an interesting account of the lowering of the section of Seattle known as Denny Hill. Portions of this hill have been cut down nearly 100 ft. The operations at Seattle have been facilitated—and in fact they would scarcely have been feasible otherwise—by the use of the hydraulic sluicing method for moving the earth, and favorable opportunities for using the sluiced material to fill low areas not far distant from the hill.

Possibly very little of this extensive regrading at Seattle could have been avoided had the founders of the city, and the city officials in the earlier days, foreseen future developments and adapted the street plan of the city to them. However this may be, there is no question but that the lay-out of the city might have been vastly improved in many of its other sections had the streets been made to conform to the topography or contour lines, instead of having been laid out for the most part on a rigid rectangular or checker-board system. Standing on one of the many vantage points which the hills of the city afford, a person cannot but be impressed with the ugliness caused by the present rectangular street plan. A few of the more progressive developers of real estate have got away from the rectangular system, and produced residence sections of relatively great charm by making their streets conform somewhat to the topography.

It is a pity that a city which has such a magnificent natural site, with such commanding views, as Seattle should have lost such a great opportunity to lay out its streets to conform with the natural features. Of course, comparatively little of this could have been done in the important business sections of the city; but in the residence sections there is no reason why curved lines and easy grades might not have been used for street lines, instead of running the streets up hill and down, always at right angles with each other, necessitating many ugly gashes in order to ease off some of the worst grades.

Progress in formulating building laws in San Francisco seems to be backward. The Board of Supervisors there has just amended its newly adopted building law by increasing the lot area percentage which may be occupied by tenements from 70% to 80%. This change was made against many protests, it seems chiefly on the demand of speculative real estate interests. The same interests made the claim that within the fire limits bay windows should not be restricted to 16 ins. projection, as the law provides. This claim too was successful, the Board changing the limitation to 3 ft. As abstracted in part in our issue of Feb. 10, 1910, the new San Francisco law is none too good; the changes seem to be making it worse.

The philosophical Mr. Dooley, in an essay some years ago upon great expositions, concluded that the greatest benefit that a city derived from holding a "world's fair" was that it never had to have another.

We commend this wise counsel to the real estate "boomers" of San Francisco, New Orleans and San Diego. These three cities are now engaged in a lively controversy as to which shall have the honor of holding an international exposition to celebrate the opening of the Panama Canal in 1915. At present San Francisco appears to have the lead. At a convention of the chambers of commerce of California cities, held at Santa Barbara on March 22, resolutions were adopted requesting Congress to appropriate $5,000,000 for the exposition, the State of California to provide a similar sum, and at least $5,000,000 more to be raised by San Francisco.

We believe that money raised from the public by taxation should not be used in this way. The current argument for all such proposed expenditures is that such an expenditure on a world's fair is justifiable as a means of public education. There was undoubted force in such an argument with relation to the expositions held 25 to 50 years ago; there is force in such an argument still, in some of the newer countries of the world; there may be some force in such an argument with relation to expositions of moderate size, especially those which are devoted to some especial field of art or industry. But the huge world-wide exposition has degenerated at the present day to little better than a fraud upon the public taxpayers who provide the funds, and the exhibitors who are, by one method or another, compelled to take part in it.

We believe this is the general opinion of those who have carefully studied such great expositions as those at Chicago, at St. Louis and at Paris. The crowds who attend these great expositions are always fewer than the promoters expect and the great bulk of those who do attend as Mr. Dooley well described, spend their time on the "Midway" or "the pike" and give only a passing glance at the elaborate and costly displays of art and industry whose educational influence, is supposed to justify the great outlay.

We are well aware that such conservative counsel as we are uttering finds little heed in these days of extravagance. Probably San Francisco will go ahead with her ambitious plans for a world's fair, and probably the taxpayers of California will have to help foot the bills. It is much to be hoped, however, that Congress may divide the $5,000,000 appropriation by 5 or 10 before it commits the Federal Government to participation.

The improvement of rails is a matter which has received considerable attention on the part of engineers and engineering associations, and for several years the quality of rails has been a constant cause of complaint on the part of railway engineers. It has been held consistently that the mills could and should furnish rails of very much better quality than those ordinarily furnished. One aim of the several associations has been to secure the adoption of more stringent specifications, but it is a well-known condition that mills have declined to furnish rails under the specifications advocated by railway engineers. In order to show the necessity for better conditions of rail manufacture, the rail committee of the American Railway Engineering and Maintenance-of-Way Association last year conducted a careful enquiry as to rail failures, and a special report on the result of this enquiry showed a great number and an astonishing character of bad rails, with all kinds of pipes and other defects. The mills claim that many failures are due to exceptionally heavy loads and severe conditions of service, but no traffic conditions can account for inherent defective quality of rails.

At the annual meeting of the above association

just held in Chicago a report was presented by this committee calling attention to some of the changes in specifications which are necessary in order to secure good rails. The report recommends the maintenance of better relations between the carbon and the phosphorus and manganese; as the present practice results in rails that are soft and do not wear well. As a supplement to this, it is recommended that both minimum and maximum deflections under the drop tests should be provided, in order to eliminate rails which are too hard (or brittle) and too soft respectively. It has not been determined as yet, however, what the limits should be. Furthermore, the committee points out that physical and chemical tests should be prescribed, and care taken to see that the material meets the requirements of these tests. This idea was carried somewhat farther in the discussion, when it was shown that in some cases the metal in the rails varies from that in the test ingot to a much larger extent than is desirable or permissible. This is a somewhat new line of investigation, and the committee was instructed to take the matter up in its work for the present year, as was noted in our report of the proceedings. An abstract of the report is given elsewhere, together with the proposed specifications for rails of Bessemer and open-hearth steel, but we may call particular attention to three significant statements in the report:

We do not know of any railway company that has succeeded in buying rails during the past two years according to a specification entirely satisfactory to the railway company. We believe that all of the specifications under which rails have been rolled have been compromises on the part of both parties, with the general result that neither party is entirely satisfied.

The railways should not be asked to take anything but sound material in their rails. The mills can furnish such sound material if the proper care and sufficient time are taken in the making of the ingots. Information derived from the tests being made at the Watertown Arsenal shows definitely that sound rails cannot be made from unsound ingots, and that, therefore, the prime requisite in securing a sound rail is to first secure the sound ingot.

The results so far obtained from the heavy base American Railway Association sections are disappointing, as we have received from the mills some rail of the new section which was as bad as that we received of the old Am. Soc. C. E. section, showing that the quality of the rail does not depend entirely upon the section.

A bill to establish a bureau of mines in the Interior Department has passed the House of Representatives and has been favorably reported to the Senate by the Committee on Mines and Mining. The proposed bureau would take over the work now being carried on by the Technologic Branch of the U. S. Geological Survey. The duties of the bureau, as defined in Section 2 of the bill, are

to make diligent investigation of the methods of mining, especially in relation to the safety of miners, and the appliances best adapted to prevent accidents, the possible improvement of conditions under which mining operations are carried on, the treatment of ores and other mineral substances, the use of explosives and electricity, the prevention of accidents, and other inquiries and technologic investigations pertinent to said industries.

The establishment of this bureau is urged by the American Mining Congress, which has among its directors Mr. D. W. Brunton, President of the American Institute of Mining Engineers. The work proposed for the bureau is really nothing more than is already being carried on to a considerable extent as a branch of the Geological Survey work; but as these investigations are foreign to the main purpose of the Geological Survey, the legislation to place them under an independent bureau seems justified.

It is well understood, of course, that under the federal system such a bureau could have little or no legal authority over mining operations carried on in the several states. As a means of public education, however, the bureau can undoubtedly do a most valuable work, and the fact that leading mine owners in all branches of the mineral industry are urging its passage is a good indication of the need for such a bureau.

It takes some cities many years to make up their mind what to do in the way of improving their water-supply, and the arguments for immediate action have to be reinforced in some cases with several typhoid outbreaks. Such has been the experience of Minneapolis. Years ago the filtration of the water of the Mississippi River was recommended by competent engineers. The subject has been up and down numerous times since then without final action, and meanwhile there have been at least two outbreaks of typhoid fever, to say nothing of the cases from month to month which have attracted less attention. Not long ago a large and representative citizen's committee was appointed to investigate and report on changes in the water-supply. It concurred in previous recommendations for filtration, but even this seems to have failed to settle the question. Sufficient sentiment existed in favor of the supply from Mille Lacs, some 80 miles distant, to lead to further delay in order to investigate this source. A report by Mr. Rudolph Hering has just been made, which shows that the supply would not only be far more costly than filtered Mississippi River water, but that it would also be limited in quantity, and would also require filtration at least at times.

A somewhat similar long delay, caused largely by agitation in favor of other sources than the near-by filtered water, occurred at Providence, R. I., some years ago. Similar delays are now taking place at Trenton, N. J., and apparently for no better reasons than those which prevailed at Providence and Minneapolis. Doubtless there are many, and perhaps diverse, local explanations for these delays, but one of the chief causes for delay is probably the popular feeling that it is an easy and less expensive matter to secure a supply which does not need filtration, even from a far-distant source, than to filter water which exists in large quantities close at hand.

Every year's advance in water filtration tends to weaken the argument in favor of the distant unfiltered source of supply, particularly in view of the fact that we are now rapidly coming to the point long ago reached in England and Germany, where the surface supply without filtration will not be tolerated in any event. When filtration is insisted upon whether or no, a large part of the argument in favor of the distant and popularly-conceived-of pure supply will fall to the ground.

The advent of hypochlorite of lime as a water disinfectant will doubtless hasten this day, but it should, of course, be remembered that it is applicable to the distant and little polluted, as well as to the near-by and greatly polluted, source of supply. This, fortunately, makes for flexibility and tends to put the final decision as between various sources of supply almost wholly upon comparative cost of construction and of annual charges.

There is a growing and commendable tendency for the state universities of the country to get into close touch with the life and work of the people of their respective states, and to cooperate with the various state departments in the securing of information and the execution of experimental and other work. An interesting instance of this sort is noted in the report of Mr. James C. Wonders, State Highway Commissioner of Ohio, for the year 1909. Mr. Wonders states that, an appropriation of $5,000 having been made available for preparing a state highway map, he at first proposed to rent and equip rooms and gather a staff of draftsmen for carrying out the work. Finding that the appropriation would be insufficient for this, he secured the cooperation of Prof. C. E. Sherman, of the Civil Engineering Department of the Ohio State University. As a result, the map-making work was carried out by the aid of undergraduates during the summer vacation, making use of rooms and other facilities at the university. There is almost no end of similar cooperative work that might be done throughout the country.

We have frequently referred in these columns to the great difficulty and expense which will be involved in establishing successful freight terminals on the banks of the Mississippi River. Some of these difficulties are well stated by the St. Louis "Republic" in an editorial comment upon the paper by Mr. H. McL. Harding, published in our issue of March 3.

Mr. Harding fails to consider two of the most important forces which the Mississippi River engineer has to reckon with in designing waterside structures—bank cutting and ice. True, he mentions the former, but in a way which shows that he is unfamiliar with the manner of its operation. He says, in speaking of the erection of terminal structures for small landings: "There are numerous situations protected by points of land from the action of the current, and generally these would be used."

This is totally misconceived. The Mississippi is heavily silt-laden, and a landing "protected by a point of land" would shoal up and become impracticable for boats. Then the points are always on the shoal side of the stream. All the deep water in the Mississippi is in the concave bends against the bank and in the narrow crossings. The bend, of course, ends in a point, but the cause of the point's existence is the swinging of the current away from the bank, which means that silting up is in progress just below it. Mr. Harding was thinking of the points in clear-water rivers, made by old gravel bars—which are essentially different phenomena.

Not until the river is revetted so that its channel will stay where it belongs and its banks become permanent features of the landscape will it be safe to build permanent terminals at any points of its course except a few, where it comes to bluff contact.

Just a year ago, in time of high water, the river was cutting its banks at a point in Northern Arkansas, and a mattress was sunk as an emergency measure. It went down in 90 ft. of water. Eastern engineers writing of the Mississippi fail to comprehend the magnitude of the problem.

The factor of ice and ice movement is ignored also by Mr. Harding. A glance will show the inapplicability of his terminal plan, as sketched, to the St. Louis levee on account of the failure to consider this problem. In the lower reaches of the river, ice is negligible; but above Memphis, every waterside structure must be built to resist its ravages. Reinforced-concrete walls and a "whale-back" model at the upstream end of the structure, so that the ice will run up harmlessly and break itself up, are absolutely necessary. The form of construction employed by the designers of the admirable New Orleans wharves would be a costly mistake at this point or at any point within 400 miles below us on account of ice.

Does Electric Current Disintegrate Concrete?

Until very recently our knowledge of the behavior of concrete, through which electric current was made to pass, has been limited to merely recognizing two important phenomena. We have previously noted (Engineering News, Dec. 24, 1908, p. 718) that (1) a disintegration, a rotting, of the body of the concrete accompanied comparatively large direct currents, and that (2) imbedded iron or steel is dissolved when the current passed from the iron to the concrete, rust being formed and more or less disseminated.

There has been much conjecture but no data to show whether this disintegration of the concrete was by electrolysis or was by indirect thermal action of the current (as mechanical expansion) or was a chemical attack of by-products of the electrolysis.

It was not known, until recently, whether or not the disintegration of unsubmerged concrete and the electrolytic corrosion of imbedded iron was similar to the phenomena noted when tests were made with material under water or brine. Formerly six lines of investigation were open, by which the much desired information, on these and allied topics, could be secured. These lines were: (1) the influence of varying the character of the concrete, (2) the effects of surface treatment to produce imperviousness, (3) the influence of surface coatings of tars and bitumens, (4) the action on independent barriers across the path of ground currents, (5) the effect of adding alum and other chemicals for the purpose of increasing strength or decreasing permeability.

The account of experiments made at the Rensselaer Polytechnic Institute and printed elsewhere in this issue, throws the first light on some of the questions we have previously raised. The number of the experimental studies and the consistent repetition of main results seemingly establish two facts. However, it is well to bear in mind that the complete corroboration has not yet been given, and it is possible that minor and secondary effects of the electric current may yet be found to limit or qualify these findings.

In the first place, the disintegration of con-

crete by large currents seems to be due to the release of heat, as we have suspected. The "rotting," near the electrodes, then, is only failure by over compression. Exactly similar results have now been obtained, on duplicate concrete specimens, by externally heating the projecting metal.

In the second place there seems to be no difference in the *nature* of the action of large currents on concrete exposed to air or to water or on concretes of different character, in some of which we might suspect the disintegration would be marked or changed.

The supposed dangers to which concrete foundations can be subjected are greatly lessened, then, if the rotting of the concrete is but mechanical failure through local over stressing and not a chemical or electrical effect of the current. In all the experiments, of which we have seen the results, comparatively large currents and high current densities (amperes per square unit of conducting cross section) have been necessary to secure the temperature effects that have been noted. Such currents and such current densities are extremely improbable in any actual case of reinforced-concrete construction for they would probably come through the chance grounding of some active electric-power supply circuit and any heavy leakage should be detected quickly, either at the point of trouble or at some control station. Straying earth currents, or deflected railway currents would not have sufficient available potential back of them so that their magnitude would be dangerous *to the concrete.*

The suspected danger of corroding and weakening the all important imbedded metal, however, does not diminish. This follows well defined laws of electrolysis, and is as serious with a minute current flowing for months and years as with a large current flowing only a few hours or days. Indeed, the dangers of the minute current are greater, for its actions are more insidious and its pressure harder to detect. As we have previously remarked (Engineering News, Dec. 24, 1908, p. 719) where any evidence of current in reinforced-concrete structures is found, every reasonable effort should be made to remove chance electrical contact from the imbedded steel and, if the structure is buried, to insulate the concrete from ground water. Here, if anywhere, "an ounce of prevention is worth a pound of cure," and the designer and contractor are the doctors.

The escape of electric current from steel structures to earth has been detected under a variety of conditions and in most such cases localized and serious corrosion has been found. The most common instances of such electrolysis have been, by far, with underground pipes and the concentration of study on such cases has perhaps diverted attention from other situations. Sometimes there is the chance contact of steel-framed structures with piping systems which are bonded to electric-railway track or which cut across track detours. Anchor bolts for elevated-railway supports have been destroyed. Such chance currents, given a poor path and a good one, do not desert the one for the other but divide inversely according to the resistances. If there is any current leaving to the earth there is bound to be some corrosion. When such trouble is known or suspected the ineffectual demand has been to insulate—insulate piping and conduit from the frame; insulate the frame from the foundation; and insulate the foundation from ground waters. These things are not usually easy to do, but a device is shown elsewhere in this issue which makes simple the insulating of anchor bolts from foundations. The bolts are surrounded by vitrified clay and the merits of the arrangement are its strength and simplicity.

The tendency, in preventing the rapid deterioration of buried pipes, also is toward universal insulation rather than toward extensive bonding of pipes to railway tracks, etc. The aim is to break up the continuity of possible electrical circuits into such short sections that the resistance of the path along the pipes is infinitely greater than that of the ground path. The use of cement joints, which are fairly good insulating connections, is becoming general on gas mains and seems to have the very desirable effect of greatly eliminating corrosion. With slightly more attention and expense, the insulating qualities of these joints can be greatly improved and the protection thereby increased.

Not much is known about the benefits of using cement and other insulating joints with water mains as their use has not been so extensive as with gas. It might be expected that their protection could never be so nearly complete, in the former as in the latter case, for the gas itself is an insulator and the water is only a poor conductor bridging the joints. In either case, the use of insulating joints should either be very general or else be at points carefully determined from electrical surveys. Otherwise unsuspected trouble may be concentrated at inconvenient points.

LETTERS TO THE EDITOR.

Credit for Methods Used in Milwaukee Street-Railway Service Investigation.

Sir: In the article in your issue of March 24 (p. 347), relating to investigations made a year and more ago by the Railroad Commission of Wisconsin with reference to the service of the Milwaukee street-railway system, I notice that the contributor who prepared this brief outline of the work failed to give credit to Mr. Ross W. Harris, of the Commission's engineering staff, for the highly original methods adopted in attacking certain phases of the problem. The substance of the article, referred to above, appeared in the annual report of the Commission recently published. Mr. Harris had immediate charge of the field observations and the preparation of the original report which was first made public by the Commission in July, 1909.

Respectfully,

W. D. Pence.

168 Prospect Ave., Madison, Wis., March 26, 1910.

The Staff System of Organization on the Union Pacific Ry.

Sir. Allow me to convey through your columns a note of appreciation to Mr. Julius Krutschnitt, Manager of Maintenance and Operation of the Harriman Lines, for the establishment and introduction of the Hine system of staff organization on railroad divisions.

All railroad men "who do things" should join in appreciation of this abolition of the dominion of the ten-cent man behind the typewriter who signs great names per.

The writer has seen divisions in a state of chaos, the company's money squandered and business at a standstill, all because "the old man" was out on the line and could not be found, and the reins were not in the hands of any one of a half-dozen competent and experienced men, but the "per" was running things.

He has seen a chap with the mental caliber of a clerk at the ribbon counter, or an office boy and the experience of "an infant in his nurse's arms" ordering men of life-long experience and recognized ability to bow down and obey him as their "lord high executioner" simply because he had been private secretary to some official farther up the line.

The Hine system will give the men who do things the show to which they are entitled and the enlarged responsibility will not enlarge the size of their hats. They have borne the burden for years and the companies are entitled to the benefits of their ripe experience.

This is from an ex-Harriman line man who hopes he may never work on a railroad again, but feels for those to whom the Hine system will give justice and opportunity.

Very truly,

J. H. Eby.

Virginia, Minn., March 20, 1910.

Formula for Crown Thickness of a Reinforced-Concrete Arch.

Sir: I note in your issue of Feb. 10, 1910, p. 171, an empirical formula for crown thickness of a reinforced-concrete arch, invented by Mr. Ernest McCullough and published by Mr. C. O. Powell. The formula is:

$$t = \frac{(20 - r)}{10} + \frac{S}{5} + \frac{75}{2\,S},$$

where t = crown thickness in inches, R and S, respectively, rise and span in feet and $r = S/R$. First allow me to ask why this formula has been complicated by introducing the letter "r" when this letter has to be explained by an auxiliary equation; the expression for t could have been written

$$t = 2 - \frac{S}{10\,R} + \frac{S}{5} + \frac{75}{2\,S}.$$

But it is hardly of any use to bring this formula down to a rational base, when it is absolutely wrong as it stands. It will be noticed at a glance that for a given span the thickness decreases with a decreasing rise, or, that an arch with a rise 1/20 of the span can be made 1.8 ins. thinner than a semicircle over the same span.

The formula is useless simply for lack of limitation. Does the inventor claim the formula adaptable for spans of 200 ft., or is it only good for very short spans, and can the ratio between rise and span be taken as low as 1:20 or smaller? In any case it is absolutely necessary to give the limits whether they are narrow or wide, otherwise a formula is useless and dangerous.

Another objection: This is supposed to be a formula for reinforced concrete, but not a word is said about the reinforcement; is this to be 1% of the cross-section or 50%, and where is this reinforcement to be placed?

Yours very truly,

W. Howalt.

512 West 156th St., N. Y. City, Feb. 19, 1910.

[The above letter was submitted to Mr. Powell, who in turn referred it to Mr. McCullough, from whom we have received the following letter:—Ed.]

Sir: Your letter to Mr. C. O. Powell has been referred to me. The auxiliary equation for "r" was introduced to obviate chances for error in using the formula. It is well known that in designing an arch some assumption must be made for thickness of crown. Older formulas for plain masonry arches give an abnormally heavy crown. The Weld formula is at present used by reinforced-concrete designers, but it is limited to bridges where $S/R = 8$ or 10. The formula is

$$C = \sqrt{S} + 0.1\ S + 0.005\ L + 0.0025\ D$$

in which

C = crown thickness in inches.
S = clear span in feet.
L = live load in feet uniformly distributed.
D = weight of fill over crown per square foot.

The formula derived (not invented) by me gives results coinciding with the Weld formula for $S/R = 10$. Your correspondent has discovered that as the radius decreases the bending moments increase. This discovery came from a study of the empirical formula and he doubts its truth. Buell states (Reinforced Concrete, Buell & Hill) that Trautwine's formula for arch crown thickness

does not conform to the principle governing the pressure line in arch rings, that the greater the rise (or the less the radius, R) the greater the moments, and the flatter the arch (or the greater the radius) the less will be the moments other things being equal.

Arch bridges of great span and small rise do not require reinforcement, except for temperature stresses, provided the abutments are stable.

My formula is empirical. It is intended to save the time of the designer who is benefitted if his first assumption for crown thickness is close to the final result. The only rational limitation to its use is the experience and common sense of the user. Since it is intended as a starter and not as a final result, the criticisms of your correspondent are unnecessarily severe. His question about amount of reinforcement is out of order for this will be fixed by the designer as he proceeds with his work. As a first assumption many designers place 0.5% of steel equidistant from the neutral axis and arrive at the proper amount and final thickness of arch crown by methods fully set forth in standard texts.

The formula

$$t = \frac{(20 - r)}{10} + \frac{S}{5} + \frac{75}{2\,S}$$

applies only to bridges having 2 ft. fill over crown and carrying a 15-ton steam roller, or a uniform live load of 200 lbs. per sq. ft.; that condition being taken which gives the maximum bending moment.

Yours truly,

Ernest McCullough.

355 Dearborn St., Chicago, Ill., March 8, 1910.

The Relation of the War Department to Bridges Over Navigable Streams.

Sir: Permit me to ask for information about the bridge-raising matter that is being agitated in Pittsburg just now. The Army Engineers are holding a hearing on the proposition to require all the bridges over the Allegheny River to be raised in order to give greater headroom. It seems that the navigation interests claim that the present clearance is not sufficient at medium high-water stages. The bridge interests says that at such river stages the current is rapid enough to stop navigation, so that there is no real necessity for increasing the clearance. As you are aware, no doubt, most of the bridges in question are owned by private companies, though the city has been considering the purchase of all the street bridges and the abolition of tolls.

As the improvement of the Allegheny River has been in progress for many years, and the ultimate result of

the canalization therefore has been in view for a long time, it would seem that the War Department should have had enough foresight to require adequate clearances at the time the various bridges were built. But the curious part of the matter is that the elevations of the several bridges with respect to the river are different, and (I understand) in some cases quite widely different. How could this come about? I assume that the War Department had to pass on and approve the plans of each one of the bridges before construction. In doing so, how could the Department approve a plan showing less clearance than given by another bridge previously built; or on the other hand how could it demand a greater clearance for the second bridge than for the first? In either case the river interests would not be helped by the varying requirement, and someone among the bridge interests was discriminated against, and compelled to waste money in providing useless extra clearance.

I have always thought that the river administration by the War Department was wise and consistent. Am I wrong in this view? If not, the case of the Allegheny bridges ought to be explained; perhaps you can explain it.

F. C.

Chicago, Ill., March 10, 1910.

[We can hardly agree with the suggestion that when a bridge clearance was fixed at a certain height above the stream, say in 1875, no greater height should be required in future bridges. The engineer in charge, say in 1890, might foresee at that time that a greater clearance was likely to be demanded for river traffic and so fix a greater height, in the belief that eventually the owners of the older and lower bridge would be required to raise it.

Neither are we convinced that it would have been better for the public had a clearance equal to what is now demanded been required at the original construction of all these bridges. Consider that the daily traffic passing over these bridges all the years since they were built would have had to mount so many more feet of elevation to cross the bridge. The probability is that the added cost of moving traffic over the bridges in all this time would have amounted to much more than the cost of raising them now to a higher elevation.—Ed.]

Brickwork in Thick Chimney Walls; Perforated and Hollow Blocks Condemned.

Sir:—Referring to the editorial in your issue of Feb. 17, regarding failures of brick chimneys, inasmuch as you raise several questions I take the liberty to suggest answers to the same and explanations of the failures to which you refer.

In the matter of the paper by Prof. J. A. v. d. Kloes, on the proper composition of mortar, this has not much application to American practice. He recommends the use of trass to overcome the effect of free lime in mortar. Scarcely one bricklayer in a hundred in America would know whether "trass" means a new breakfast food or a new waterproofing dope for concrete. "Puzzolana" would also puzzle our practical builders. Yet they succeed in making brick walls that stand the test of time and that withstand a great deal of battering to break down and break up, when they are wanted out of the way.

If the kind of mortar used were the cause, and if the addition of trass or puzzolana were essential to good mortar, failures of brick walls and chimneys supporting heavy loads would be quite common in America. They are not common, however.

In 1900 a brick chimney in Hamilton, Ont., failed. This chimney had very heavy walls, being 5 ft. 6 ins. thick at the bottom, where failure occurred. The wall at the top was 26 ins. thick. Of course there is an appearance of safety and strength in a thick wall like this; but, when it is analyzed, the very excess of thickness is seen to add to the liability to failure, for reasons that will be given presently. At the base of the chimney referred to, the load per square inch on the brickwork would be about 120 lbs. The wind load on this stack would not need to be considered, even if the wall had been only a half or a third as thick. Wind played no part in the collapse, as only a breeze was blowing. Further if the wall had been only a half or a third as thick, it would have had the same unit pressure from its own weight. It is hard to see why such thick walls were made in this chimney, which was 200 ft. high and 19 ft. 4 ins. in diameter, at the base. Unless a wall has a heavy taper, or unless its weight is needed for stability against wind, or unless its thickness is needed to reduce slenderness, excessive thickness means weakness instead of strength; for, while a thick wall being less slender will apparently be capable of carrying more unit load, there is a practical consideration that diminishes its unit carrying capacity.

The practical consideration referred to is the manner in which a thick brick wall is laid. Everyone who has watched a brick wall being built is familiar with the way that a bricklayer will throw in dry bricks into the backing, probably putting a little ridge of mortar over them. The thicker the wall the more carelessly is the brick laid in its interior. It is the exception to see a thick wall thoroughly flushed with mortar.

Even if a brick wall 5 ft. 6 ins. thick were flushed with mortar the lime mortar in the interior would be a long time hardening. We are told that lime mortar in the interior of very thick walls built ages ago has been found to be still soft.

In the particular chimney referred to there were found in the interior of these fortress walls large stones in the section where the chimney cracked and sheared off. The wall was laid in lime mortar. It is scarcely likely that these stones were flushed with mortar or that the cavities were filled. It would take bucketfuls of mortar around each stone, and mortar is more expensive than air, or even stone or brick.

It would have been immensely better if these walls had been a small fraction of the thickness they were made, and laid carefully with cement mortar or even lime mortar.

The two chimneys to which you refer in your editorial had one thing in common. They were made of perforated brick. Now hollow blocks of all kinds make a good talking point. They are light; they are strong; they are thoroughly and evenly burned; they are non-conductors; etc. In an office it is easy to show that the several webs and shells make a continuous compression member, for do they not butt against one another and have a neat mortar joint where they meet? These hollow and porous blocks can be very neatly laid—in the office. Out on the job it is quite different. The brick masons are not talking—at least not about the strength of a wall made of hollow blocks, nor are they caring. They are putting up these blocks in the way their experience has taught them; they are carrying out their employers' orders not to throw away mortar by stuffing it into these useful hollow spaces. They are laying up a wall that looks nice on the outside, for there is a joint of mortar on at least two edges of the block. It is not practical to get all of these ribs just over ribs of the block below, and it is not practical to fill all of these joints with mortar without losing a lot of mortar in the holes.

An inspector would have to be very much more vigilant on a hollow-block job to insure the filling of every joint than he would on a concrete job to insure the making of good concrete, in spite of all that is said of the need of eyes on a concrete job. It is comparatively easy for an inspector to cover a large concrete job and see that good work is done in every part, but the laying of hollow blocks that take heavy pressures, as in chimneys or flat arch floors, would need an inspector for every one or two workmen.

Furthermore, the inspector who tries to make workmen do what is "never done" will find that he is up against it, especially if he is a young member of the "show me" fraternity.

It is easy to see what would happen to a wall under heavy pressure, if the blocks are bedded on two edges only, when the load is a respectable fraction of that which, in a testing machine with neat cement joints and perfectly fitting compression plates, would crush the blocks. It is not difficult to find a plausible reason for the cracking of such a wall, or for its leaking smoke even without cracking.

The account, in your issue of Aug. 19, 1909, of the wrecking of one of these chimneys states that "a large proportion of the bricks were blackened on two or more sides." This is clear evidence that the said two or more sides were never covered with mortar.

The best way to overcome such faults due to careless workmanship is to make it impossible for the workmen to commit them, by using solid bricks, or better by using concrete.

The same fault exists in flat tile arches and slabs. Hollow tiles with the "hollows" and the thin edges of the webs meeting cannot take much thrust, particularly as it is practically impossible to make a full mortar joint between the effective parts of the tile. They cannot be flushed, as the liquid mortar would be lost in the openings. It would be asking too much of a workman to expect him to put mortar on all of these webs. Further, the webs are often fire-cracked and curled.

I have long been of the opinion that a large part of the strength of the ordinary tile arch lies in the wooden sleepers and the concrete filling. In the old style of arch having 5-ft. or 6-ft. spans this was not a matter of so much consequence, as the thrusts were not large, and the sleepers would carry a large load because of the short span. But I know of one bold piece of construction in a large building where, on spans of 16 ft. and over, there are slabs consisting of a 1-in. layer of cement mortar in which steel is embedded and a layer of soft hollow tile 5½ ins. deep with ¾-in. shells and webs. The floor is laid on this. Is it any wonder that reinforced concrete builders are using higher and higher units and are being shut out of competition, when such monstrosities as this pass for engineering and are approved by city building bureaus?

Edward Godfrey.

Monongahela Bank Bldg., Pittsburg, Pa., Feb. 21, 1910.

More Concerning the Panama Canal and Other Great Construction Projects.

Sir: Conservatives owe a great debt of gratitude to Engineering News for its consistent policy of criticising dubious schemes and its opposition to chimerical plans. Its stand some years ago against the building of the New York State Barge Canal required courage and may in the future add credit to its opinions if the canal shall prove to be inefficient or inadequate to modern conditions.

It is hardly to be doubted that the country is entering upon an era of great projects—forestry, irrigation schemes, water power, "military" roads, as advocated by our friend Hobson, of Alabama, (Cong. Record, Vol. 45, p. 3037), canals and river improvements—all to be constructed by the government (state or Federal). Whether these schemes will benefit or injure the country is the debatable point.

If all argument as to these expenditures is to be suppressed by the simple process of calling all such projects "improvements" and classing those opposed to the waste of the nation's wealth as pessimists and unprogressive citizens, there is no telling into what riot of

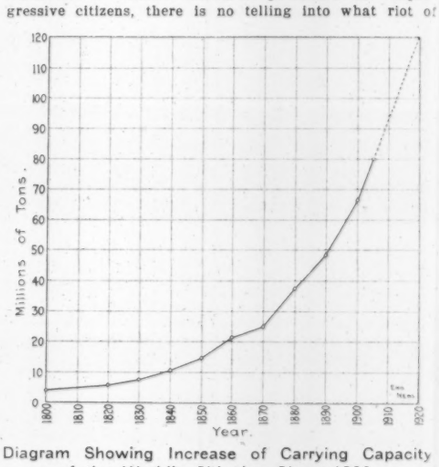


Diagram Showing Increase of Carrying Capacity of the World's Shipping Since 1800.

extravagance the nation may be drawn. Surely it is a question to which there are two sides and if those who oppose such things being done do not speak out, the matter will go by default and the waste be started by "unanimous consent," so to speak. I feel that Engineering News is an ally, if not in all cases, still in many and important instances and willing to allow and promote reasonable discussion.

You were kind enough to print my letter questioning the economic soundness of the Panama Canal enterprise. With your permission I should like to present some further figures, not available to me at the time I first wrote, as a rejoinder to your comment on my letter. My first letter was an attempt to show that it is as yet uncertain that the Panama Canal is an admirable piece of work, from the view point of economics, and that too much work should not be patterned after it, work such as making over the rivers in this country, in an attempt to make economical freight carriers of them. I realized when writing that it was too late to discuss the Panama Canal as a live issue for it is almost fait accompli, but there is still time to discuss, as Mr. John Howe Peyton did in your issue of Feb. 17, the opening of an extra big "pork barrel" in the way of river improvement work in the United States.

In your issue of March 3 is an article by Mr. H. McL. Harding giving numerous instances of the actual cost of transferring freight, a point which you, perhaps justly, criticised in my argument, saying I had put the cost much too low. From Mr. Harding's article I have taken 17 specific cases, which I will not detail here, and have averaged them, the result being an indiscriminate average of 34.6 cts. a ton as the cost of transferring freight. Applying this figure to my estimate of the cost of trans-shipping freight at Panama, we would have two transfers, say 37½ cts., for easy figuring = 75 cts. per ton, 50 miles of haul at 1 ct. a mile (by your estimate in editorial of March 3 issue 3 mills would be nearer) = 50 cts. per ton, total $1.25 per ton, which on 6,000,000 tons would be $7,500,000 or half the estimated cost of interest and maintenance on the canal as given in my former letter even allowing no higher maintenance than $3,000,000 annually.

Mr. G. A. Ballard, however, in "The Contemporary Review" for December, 1908, estimates the lowest possi-

ble working expense of the canal at $5,000,000 per annum. As bearing on the estimated tonnage to cross the Isthmus, some comparisons will assist best perhaps since the figures are so huge as to be almost unimaginable. Take then, the tonnage handled in 1908 by American railways, 1,500,000,000 tons, and it seems as if even 10,000,000 tons to cross the Isthmus were a trifle. But the ocean-borne freight has maybe 30 times the "length of haul" of the freight on railways, even of America. On the other hand the tonnage inbound and out of the great port of London is only 34,124,083 tons (in 1906); that of New York 21,855,946 tons (in the foreign trade), and that of Chicago 14,532,590. All the ships of the world have a capacity of but 40,000,000 tons. Suppose we take the total tonnage of principal ports of the Pacific coast of North and South America, which is as follows:

	Tons.
Valparaiso, Chile	3,469,000
Iquique	3,092,000
San Francisco, Cal.	1,734,429
Puget Sound, Wash.	3,281,861
Vancouver, B. C.	3,055,487
Total	14,632,777

and assume that all of this passes across the Panama Isthmus, by rail, it would cost, by above estimate of trans-shipping charges only $18,290,971, or about $3,000,000 more than the estimated interest and maintenance on the canal or $1,000,000 more if we take Mr. Ballard's figures. If we look to the possibilities of future traffic of Panama Canal, as you say in your comment on my former letter; if we discount the future, and if we make that future sufficiently remote, we can figure out any result whatever.

For instance, as an illustration of rapid growth, the increase in the world's ocean carrying trade from 1800 to 1905 has been as shown by the accompanying diagram, drawn from figures compiled by the Department of Commerce. And the world's per capita commerce has increased from $2.31 in 1800 to $13.36 in 1900—six fold. Such a rate of increase continued for a while would produce so much traffic as to throw enough traffic to the Panama Canal merely as "overflow" to make it pay doubtless.

But still, even granting that this be so, why not wait for the traffic to develop? Why spend $400,000,000 now for unproved traffic? Why take such risk when none need be taken? Why not work on the economic plan of developing the traffic first and then building to suit it? Isn't a bird in the hand worth several in the bush? Do we not see all around us people from whom even a slight lifting of burdens would be very welcome and permissive of their doing much for themselves and for others? Isn't the country full of people who could with even a little capital make for themselves a great per cent. of profit on that capital? Why handicap such people with inefficiently collected and inefficiently expended taxes to accomplish something for their good? Why assume that people do not know their own good, but must be educated, coerced, forced, given "object lessons" in road building, etc., and furnished with high salaried experts from Washington to teach them to build roads, sow gardens, "conserve" forests, measure and develop water powers, operate gas producers and gas engines, and do a thousand other things?

If Panama is not so admirable, save as a tour de force of a great nation, isn't it likely that many of these river improvement schemes are more unsound still since they are a stemming of the tide of a declining rather than an increasing traffic. Let us have full and fair discussion.

Respectfully,

Nashville, Tenn., March 12, 1910. A. P. C.

Another Plea for Breadth in Engineering Education.

Sir: There has been much discussion recently concerning the status of the engineer and particularly concerning the payment for engineering services. It has occurred to me that in the statement of the case the main issue has been obscured and that something other than mere accident is operating to foster the low opinion in which the profession is generally held.

It is the favorite method of argument to compare the fees charged and received by engineers with those of attorneys and other professional men and in the heat of indignation over the disparity, to seek outlet for the feelings in harsh words. There is evident injustice but this is a most absurd state of mind in which to determine its cause. I confess that at times I myself am prone to indulge in wrathful utterances and to sink into unreasoning criticism, but on further thought always revert to the belief that below all is some live reason. Unless the matter be approached in a spirit of calmness there can be no rational conclusion reached and so we ought to lay bare the whole affair and like the surgeon who inflicts pain to effect a cure, dissect without flinching.

It may be contended that to fix as a criterion of merit the extent of one's worldly goods or (what amounts in practice to the same thing) to compare fees and salaries, is mercenary in the extreme and a reversion to materialism; but in the end we are forced to the admission that on precisely such a basis does the world fix its estimates of value. I do not believe that engineers are less keen in their appreciation of the good things of life nor less desirous of attaining to a reasonable amount of wealth than other men. It is well therefore at the outset to admit without argument the seemingly base standard of judgment and to proceed to hard facts. I do not venture to say that all engineers are so weighed but that the principle of a balance between worth and emolument is generally applicable.

From the very nature of his employment the engineer is a rover. His frequent and often protracted absences from the community which he calls his home lead to the reasonable belief that he is a transient only and not a fixed member of the community. To the average mind (and in this I am sorry to include many ordinarily well-informed persons) the engineer is a strange person who lugs about a strange instrument setting it up and taking it down without seeming necessity. He frequents the most outlandish places for his operations. Clad in the roughest of attire, with muddy boots and a general air of disarray, he labors in rain and mud to accomplish something that any average person could accomplish without any such antics. It is the general conception that pure English is unnecessary for success in the profession and rather a hindrance to effective work than otherwise. The engineer's work does indeed frequently command the favorable attention of his fellows, but he himself remains unknown. It is poetically held that a sinking of self in a labor is the true spirit of work; but I am free to confess that upon occasion a kind word of praise or appreciation would be a welcome diversion. The picture may appear overdrawn and designed, like the scare headings of the yellow press, to attract attention; yet I am sure that, to a greater or less extent, it represents a real condition.

At bottom, the reason for the low estimate of the engineer is his lack of preparedness to enter the life of the community as a well-rounded man. For this the technical schools are largely to blame. Graduates are sent forth generally with a fair knowledge of the elements of engineering but with a weak preparation in the English branches. In consequence at the very outset, they are at a terrible disadvantage, for unless ideas can find expression in clear and straightforward language, the impression they will make, will be slight. In observing a number of young men fresh from college I have noticed that purity in diction and precision in expression are rare attainments. In fact a decided slovenliness is all too evident. In the effort to extend the curriculum to cover the widest range of technical subjects, and at the same time to shorten the period of preparation, something must be roughly sacrificed and where so excellent a chance as in the non-technical studies?

In this the colleges have missed their excuse for existence, for in the end they must justify their work by the offer of a broadening course of study. I have no patience with such colleges as seek to eliminate all else but technical requirements and the world views the matter in much the same way. Few of the graduates possess more than a bowing acquaintance with the modern languages and I doubt if in the great majority of cases, any serious consideration is given to continuation in them after graduation. Considering that since the war with Spain the relations with Spanish-speaking countries have markedly developed and that with the cementing of ties an increasing call for American engineers in such countries is bound to follow, the neglect to make the study of Spanish a requirement for graduation, seems the grossest absurdity.

For some years I was resident in a country where the language was Spanish and there was daily contact with a public which used this medium to the exclusion of the legally-sanctioned English, yet it was a common observation that engineers in spite of the manifest advantage of acquiring the language, were among the most backward of the government officials, in that direction. It was evident that the matter was treated with levity even though the immediate benefits were apparent.

In knowledge of the law too, engineers seldom show to advantage. Unquestionably a familiarity with the elements of business law is a positive necessity, especially such parts as deal with the preparation and execution of contracts, yet if judgment is to be based upon some of the contracts drafted by engineers, it might be concluded that the profession regarded the territory of law as undiscovered country. The ordinary course in business law provided by the engineers' schools is treated by the students as a huge jest and in a large sense as a time-filler.

Of the principles of esthetic design, the engineer is usually in delicious ignorance and so the charge of vandalism is sometimes richly deserved. Economy above all else seems to be the guide in most works and the advice and approval of the architect are viewed with suspicion and distaste. It is not that economy in itself is a fault, but that it should operate to destroy all other considerations, is a manifest absurdity. The relations of architects and engineers are closely interwoven and there should be peace among the professions. A compulsory course of reasonable length in the elements of architectural design should be incorporated into all engineering courses and the necessity for harmony and suitability of detail strongly impressed upon the students.

There is a certain flippancy in young engineers and a wide-spread tendency to discount the usefulness of other than strictly technical information. This follows from the lack of variety and breadth of the preparatory courses. Such a position is a confession of unbalance and is deserving of all the harsh criticism which it receives. There is nothing of discredit in the effort to seek development along various lines yet many young men so regard the effort. It may be answered that this is only the inexperience of youth, but as the thing often continues beyond the youthful period, it seems to be a serious defect in the educational training.

Rarely is the engineer a social animal. The demands of society meet with indifference from him and the community loses interest in him. Long residence in the field where formality disappears, tends to blunt the finer qualities and society tires after a period of accepting the stock excuses. A transfer of field manners into ordinary social life will not be long endured and neglect of the social courtesies will not be condoned. In politics and the great civic movements the engineer seldom occupies more than a professional position and his influence is practically nil. In these activities he as much as any man should exert a strong influence and should throw his weight into the contest. The state has a right to expect his interest and assistance. It is but a poor defense that politics is full of graft and dominated by unscrupulous men, for no hope for betterment can exist unless all good citizens are moved to righteous indignation and lend their aid. It is a refusal to meet the issue squarely and in the final judgment is a confession of a want of public spirit. We cannot assume the cloak of public benefactors and then dodge the public burdens.

It may be asserted without serious danger of contradiction that as a class engineers are found with clean hands. Beset with all the temptations of self-interest, by the advances of unscrupulous men and by the largest opportunities to break their trust, they emerge from the fight without discredit. This reputation is to every one of us a source of pride and satisfaction. But it should not blind us to our deficiencies or make us self-sufficient. The needs of his community and the desire to be regarded as a worthy member of it, should impel the engineer to exert all his force to increased action. He needs to mingle more with men, to join in their thoughts, to ally himself with them in their work of civic reform, to avoid the snares of the narrow technical path, and above all to admit that there is something in life beyond mere technical detail. The hope of better things lies in the colleges and at this time when the engineering courses are being subjected to rigid examination, it is pertinent that we realize their failings and be in a mood to assist in corrective measures. The pitfalls of greater specialization must be avoided if the profession is to inherit its rightful position in the world. We are useful to the world but we lack influence.

Robert G. Dieck.

614 Corbett Building, Portland, Ore., Feb 15, 1910.

Notes and Queries.

A recent passenger elevator accident in New York City, in which the traveling sheave frame was broken so that the car fell, has led to the suggestion that the Building Bureau make a special inspection of the strength of these frames before approving elevator installations. These frames are often made of forged straps, and it is proposed to require special reinforcing of the loops of such strap frames. The Bureau is desirous of obtaining suggestions for such a specification or any related suggestions tending toward increased safety of elevator installations. Mr. R. P. Miller, M. Am. Soc. C. E., the Superintendent of the Bureau, may be addressed at 220 Fourth Ave.

THE TRANSSIBERIAN RAILWAY.—According to a report in "Le Genie Civil" the Russian government has commenced double tracking the transsiberian railway and has also started a cut-off between St. Petersburg and Omsk, via Viatka and Perm, which will reduce the total length of the line about 166 miles. At the same time the Russian and Chinese governments are now engaged in the construction on the new transmongolian railway, which will connect Myssovaja, on the transsiberian road to Pekin, via Kiahta and Kalgan. This new line will remove the necessity of passengers from Europe to Pekin going through Manchuria, as has been heretofore the case.

When all these new lines are in service, the total railway distance from Paris to Pekin will be 6,320 miles, instead of the present 7,452, and the traveling time will be 9½ days instead of the present 14. The mean speeds maintained will be 37 miles per hour from Paris to St. Petersburg, 26 miles per hour from there to the Chinese frontier and 17 miles per hour in China.

"Reasonable Rates" for Gas and Electricity as Determined by the Wisconsin Railroad Commission for the City of Madison.

[Contributed.]

The Railroad Commission of Wisconsin on March 8 handed down its decision in the case of the State Journal Printing Co., et al, vs. the Madison Gas & Electric Co. This case was one of the first to be tried under the Wisconsin Public Utilities Law, and called for an interpretation of practically every question affecting the determination of "reasonable rates" as required by that law. The decision disposes of many questions as to just what elements enter into "reasonable rates" for gas and electric service and will probably affect the determination of many public-utility rate cases now pending before the Commission. In short, reductions were made in the company's present rates amounting to 2 cts. per KW.-hr. for the majority of electric-lighting consumers, and 10 cts. per 1,000 cu. ft. for the majority of gas consumers. Conditions of gas and electric service at the present time were found to be satisfactory and to compare favorably with conditions found in other cities in the state.

The company's rates at the time complaint was filed, were as follows:

GAS.	Gross.
First 1,000 cu. ft. at	$1.50
Next 4,000 cu. ft. at	1.40
Over 5,000 cu. ft. at	1.25

Each of said rates being subject to a discount at 25 cts. per 1,000 cu. ft. if paid each month by the tenth day thereof.

ELECTRIC.	Watt-hrs.
16c. from	0 to 25,000
14c. "	25,000 to 50,000
12c. "	50,000 to 78,000
10c. "	78,000 to 200,000
8c. "	200,000 to 500,000
7c. "	500,000 and over.

Power, from 5 to 8 cts. per KW.-hr.

Upon the much-mooted question of how to assess capacity costs upon separate consumers equitably and in a practical manner, the commission considered and abandoned the total connected-load basis and the number of rooms basis, holding that while these forms of assessing capacity costs might prove equitable in some cities, conditions in Madison do not permit their use there.

Some 4,000 monthly bills of consumers were analyzed in detail and the seasonal variation in the sales noted, together with the amount of current consumed and the variation in installation, and also the variation of maximum demand to connected load as obtained from data secured in Madison and other cities. The investigation was perhaps the most thorough ever undertaken in this country with particular reference to consumer data. The new rates ordered are as follows:

GAS.—$1.15 net or $1.25 gross per 1,000 cu. ft. for the first 2,000 cu. ft. used during any one month through any one meter. $1.05 net or $1.15 gross per thousand for the next 3,000 used during the same month through the same meter. 90 cts. net or $1 gross per thousand for all gas in excess of 5,000 cu. ft. during the same month and passing through the same meter.

ELECTRICITY.—Primary rate; 14 cts. net or 15 cts. gross per KW.-hr. for current used equivalent to or less than the first 30 hours used per month of active connected load. Secondary rate; 8.5 cts. net or 9.5 cts. gross per KW.-hr. for additional current used equivalent to or less than the next 60 hours' use per month of active connected load. Excess rate; 5 cts. net or 6 cts. gross per KW.-hr. for all current used in excess of the above 90 hours' use per month of active connected load.

The "active connected load" shall in each case be a fixed percentage of the total connected load, consisting of lamps, appliances, etc., installed upon the consumers premises.

Class A includes residences, dwellings, flats and private rooming houses. Where the total connected load is equal to or less than 500 watts nominal rated capacity, 60% of such total connected load shall be deemed active; where the installation exceeds 500 watts nominal rated capacity, 33⅓% of such part of the total connected load over and above 500 watts shall be deemed active.

In Class B, (business houses, etc.) 70% of total connected load shall be deemed active.

In Class C, 55% of total connected load shall be deemed active. This class consists of federal, state and county buildings, churches, missions, hotels, clubs, factories, shops, schools, stables and garages, warehouses, etc.

In Class D, 30% of the total connected load shall be deemed active. Such class shall consist of the University of Wisconsin.

In Class E, 55% of the total connected load shall be deemed active. This class consists of all interior lighting for the City of Madison, including commercial current for schools, police and fire stations, libraries, hospitals and other city buildings.

In Class F, the total connected load shall be deemed active. Such class shall consist of unmetered lighting for signs, outlines and windows contracted for upon a yearly basis.

The minimum bill for any class shall be $1 net per month. A discount of 1 ct. per KW.-hr. shall be made upon the gross rates for prompt payment.

Company shall renew burned-out or badly dimmed carbon-filament lamps of the type originally furnished by the company when returned unbroken to its office.

For the reconnection of meters for the same consumer upon the same premises, a charge of $2 is deemed reasonable.

COMMERCIAL POWER.

For direct-current electric-power purposes as measured by meters owned and installed by the company, a charge of 50 cts., net, per active horse power capacity per month, plus 4 cts., net, or 5 cts., gross, per KW.-hr.

"Active horse power" shall consist of a fixed percentage of the nominal rated capacity of the motor as indicated on the manufacturer's nameplate. The following percentage of such capacity shall be deemed active:

	Percent.
Installations under 10 HP. and only one motor is used	90
Installations under 10 HP. and more than one is used.	80
Installations of 10 HP. or over and less than 20 HP. irrespective of number of motors	70
Installations of 20 HP. or over and less than 50 HP. irrespective of number of motors, yearly contract basis	60
Installations of 50 HP. and less than 100 HP.	55
Installations over 100 HP.	50

In the last three cases, if less than yearly contract basis, 70% shall be considered active. The minimum bill shall be $1.50 per month.

Method of Valuation.

The reproduction values (new and existing) of the property of the company as determined by the engineering staff was, for the gas property, $402,834 and $338,961. For the electric light property the values were $524,224 and $419,189. To these values were added for working capital $8,931 to the gas and $10,613 to the electric properties, making the total valuation of the combined plants, $946,602 new and $778,694 existing.

The Commission concluded that the company is entitled to a reasonable return in the way of interest and profits on a valuation of about $412,000 for the gas plant and about $535,000 for the electric plant. It stated that consideration has been given, in arriving at this value, to other factors than cost of reproduction new; the cost of reproduction new less depreciation, the original cost, book value, capitalization, gross earnings, operating expenses and net earnings, the cost of building up the business as shown by the value of the gas plant when computed on a 7½% earning basis and by the value of the electric plant when computed on an 8% basis and by the value of both plants when figured on an 8% earning basis. These several topics are discussed in what follows.

REPRODUCTION COST.—The method of arriving at the cost of reproducing the plants is shown in one paragraph of the decision.

The reproduction cost new of each item was taken, based upon accumulated figures secured by the staff from a variety of sources, from the cost of similar work practically done, the experience of men in previous work, always guarding against extreme fluctuations in market prices, and by adopting a five-year average basis for prices wherever practicable. The result arrived at by the staff, therefore, would not show the minimum figure at which the plant might be reproduced, but would be a liberal treatment of the property. After working along this line the staff had a series of conferences with the engineers of the company, the tendency of which conferences was to change some of the unit prices and make an important addition in the inventory. The final revision offered by the Commission's staff gives a valuation of respondent's property that on the whole may be regarded as liberal.

REAL ESTATE.—The most disputed questions in the matter of valuation of the company's property were over the question of real estate and intangible values. The company introduced much expert testimony as evidence of the value of its real estate and devoted much criticism to the methods advocated by the engineering staff of the Commission for determining this value by comparing the ratio of recent sales to the assessed value of the property for purposes of taxation. The Commission discounted much of this testimony of real-estate men concerning the offers which had been refused; concerning these, it was stated:

Offers of this kind are hardly the best testimony concerning values as they are seldom made in such form as to be susceptible of verification.

After careful consideration of the "sales method" as compared with the "local expert" method, in this and other states, the Commission reached the conclusion that, its original estimate had to be raised, but defended its work by stating that

the method used by the engineer and his staff in determining the values of real estate shows that these methods when properly employed are scientific and lead to correct conclusions. The engineers of the staff have employed these methods very extensively in the valuation of real property in all parts of the state and generally with satisfactory results. The engineers are also unbiased in their attitude toward the proceedings, such as those involved here. They are witnesses for neither petitioner nor respondent, and have only one aim in mind, viz., that of arriving at the truth through the application of correct methods of investigation, and their experience and training entitle them to be classed as experts in such matters. It is, of course, a fact that the determination of the market value of any piece of real property is ultimately a matter of judgment, and that no method of valuation yet discovered will disclose the exact value or do much more than indicate within fairly narrow limits the figures at which the value should be placed. But it is believed that the methods employed by this staff are the best that have thus far been used for this purpose.

The method of determining the value of the land, by comparing the ratio of recent sales to the assessed value of the property for taxation, consists in examining the records on file with the recorder and getting, from these records, the assessed value of land similarly situated. The value can be determined for the plot in question at either so much an acre or so much per front foot. Recent sales are then consulted and a ratio showing the relation between the assessed value to the selling value determined. A number of these sales, where bona fide, furnish a basis for arriving at a percentage which can be applied to assessed value to determine probable market value.

The above method was tested by Mr. Thos. A. Polleys, Secretary of the Chicago, St. Paul, Milwaukee & Omaha Ry., in the city of St. Paul, Minn. Mr. Polleys chose a district comprising over 300 platted blocks, extending from Western Ave., on the east to Cleveland Ave. on the west (a distance of 3½ miles), and from Marshall Ave. on the north to Osceola Ave. on the south (a distance of nearly a mile). This district was selected that the great activity in real estate for several years preceding should insure ample sales data, and also because of the representative characteristics of the territory. The process was directed to the determination of the correct "ground" value per foot, exclusive of improvements, in a given year (1907). Mr. Polleys performed this work himself chiefly and then charted his own determinations and submitted duplicate copies of the same to some 14 highly qualified expert real-estate dealers in St. Paul. These were urgently requested not to be influenced by the figures which he had placed on the properties, and also to give figures only on those properties where they had direct personal knowledge of their values. Some 12 of these experts submitted more or less complete responses, and an average of their figures showed that the tendency of the sales method was to give a deficient result averaging 3.8% lower than the average result of the local real estate experts. This was strikingly confirmed by another instance consisting of the valuation of the terminal land of the Chicago, Milwaukee & St. Paul Ry., at Milwaukee, in 1903, under the auspices of the Wisconsin Tax Commission. The market value of these lands, consisting of 500 acres in the aggregate, scattered through some 15 wards or assessment districts of the city, was approximately $6,000,000, and the value of the lands as placed by the Commission according to the sales method differed only 3.5% from the result as determined by the specially qualified local real estate experts. It would appear that on the average the "sales method" is about 4% lower than "local-expert method" in the result obtained.

PAVING.—The engineers, in their valuation, included the extra cost that would have been involved if the gas pipes had had to be laid through paving or after the streets had been paved. This amounted to about $4,000. The commission held: that whether this cost should be included in the valuation of a plant when determined for rate-making purposes is a question that would depend largely upon whether the expense was actually incurred; that is, whether the pipes were laid before or after the streets were paved.

If the pipes were laid after the streets were paved, then it would seem that the additional cost should be included. If, on the other hand, the pipes were laid in advance of the paving, then equity as between investor and consumers would seem to require the omission of all or the greater part of this cost from the valuation.

CONSTRUCTION ALLOWANCE.—The engineering staff allowed 12% for engineering, inter-

est and contingencies. The company claimed 18%. Practically all the witnesses in the case placed the cost of engineering at about 5% on the cost of the physical plant and there was no division in the testimony as for as this item was concerned. The staff used 4% for the rate of interest during construction, the company contending for 6%. The staff first used 3% but changed to 4% because it was estimated that the construction might require more than one open season. Interest at 3% under a one-year construction period would be at the rate of 6% for the whole amount of capital during half of the period of construction.

LEGAL WORK, INSURANCE AND CONTINGENCIES.—The staff allowed 3% on these items. One witness testified that he was in the habit of allowing 4% on these items. The other three witnesses declared that 7% would be necessary, and divided same as follows: 2% omissions and contingencies, 3% legal and organization, 2% casualty liability.

COST OF PIECEMEAL OVER CONTINUOUS CONSTRUCTION.—Experts for the company claimed that the additional cost of piecemeal construction over continuous construction would be not less than 15%, and claimed an allowance of $143,455 to reproduction cost, or $116,291 to present value on this account. The engineers of the Commission maintained that the inventory valuation was sufficiently high to cover this alleged extra cost, as in making the valuation each item had been taken separately as though purchased in the open market, and no consideration was had of the fact that the company or contractor would secure special figures in purchasing larger quantities of material equipment and labor.

WORKING CAPITAL.—The company claimed that an allowance of $100,000 would be necessary to be held as quick assets or working capital, either as money available or as stores and supplies on hand. The inventory as made by the engineer covers $30,130 present value of stores and supplies. The method of estimating the amount of working capital required varied according to the testimony. One witness stated that frequently one-third of the yearly receipts was adopted to estimate the amount of working capital. In this case the witness stated that 10% of the total capitalization should be kept liquid. Another basis mentioned was to use a certain number of cents per 1,000 cu. ft. of gas sold. He had used 20 cts. and sometimes 30 cts. per 1,000 cu. ft., and considered 30 cts. reasonable under general conditions.

Another witness considered the working capital to consist of the difference between the quick assets and the liabilities when the former were the greatest. That is, the working capital would consist of the amount by which the current or quick assets, as distinguished from the fixed assets, exceed the sum of the current or quick liabilities as distinguished from the fixed liabilities. The commission stated:

> This is no doubt true, but these facts show the existing condition of a plant in this respect rather than the amount that is actually required for working capital. A working capital of even less than 15% of the amount derived from the sales of gas and current, or from $45,000 to $50,000 is fully adequate under present conditions.

VALUE OF INTANGIBLES.—The company contended that an important element in the valuation for the purpose of fixing reasonable rates is of an intangible character, variously designated as "Going Value," "Going Cost," "Development Value," "Development Cost," "Cost to Get the Business," "Enhanced Value Over Physical Value Arising Out of Connected Business," and numerous other names. Several expert witnesses testified on behalf of the company.

Dr. A. C. Humphreys, President of Stevens Institute of Technology, estimated the intangible value of the property at $300,000 to $310,000, basing same upon the assumption that the development value was equal to not less than one-third of the value of the physical plants. Another method of estimating going value which he stated was used, was to base the same on the amount of business actually secured, and the value of the business actually secured would be equal dollar for dollar to the gross revenue; this is, if the gross revenue were $300,000 a year, he would place the intangible value at not less than $300,000.

Mr. A. E. Forstall, Consulting Engineer, New York City, placed the intangible value of the plant at $250,000. He would take into account the possible earning capacity of the plant, the value being the sum upon which the plant could be made to earn 10% after taking care of depreciation, repairs, operating expenses, contingencies, etc. He would also take into account other matters of operating conditions. He was of the opinion that each customer connected with the plant was worth some $30 of going value.

R. B. Brown explained the term "going value" as the money which had in all probability been actually spent in order that the structurally complete plant might be transformed from a mere assemblage of physical parts into a going, operating, service-giving, revenue-producing establishment. Many successful managers are willing to spend dollar for dollar for new business. Most managers will spend not less than one-half the first year's income in order to get the business. His estimate was $250,000.

After careful consideration of the financial condition of the plants in question, the Commission concluded that they had no going value over and above the cost of reproducing these plants new. The Commission states, however, that

> while public utilities, under both the Common and Statute law and under normal conditions, are entitled only to reasonable returns on the investment, justice as well as equity appears to demand that the amounts, if any, by which they have failed to earn such returns should be considered in fixing values and rates for such plants. In fact, such consideration would in most cases seem to be absolutely necessary in order to secure the capital required, for it must be obvious to all that unless the prospects of obtaining at least a reasonable amount for interest and profits are at least fairly good, private capital will not enter such enterprises. By this is not meant, however, that deficits from operation can be equitably taken into account in the appraisals or rates regardless of the conditions under which they are incurred. As already stated, when such deficits are due to abnormal conditions or due to bad management, defective judgment, extravagance, lack of ordinary care and foresight, unduly high capital charges and other questions of this nature, it is manifestly clear that they should be accorded little or no consideration in either the valuation or the rates.

To determine the earning value on the company's plant, the Commission proceeded as follows.

The cost of the plants at the time they were secured by the present company were ascertained as closely as possible. This amount, with the first years' expenditures for extensions and with allowance for increase in real estate values, for depreciation and reasonable profit, was charged against the plant. The plant was credited with the earnings of the year and the balance was taken to show the value of the plant at the end of the year on which a reasonable return should be expected. Such calculations were made from 1896 to 1908; two such are shown below for the electric plant alone.

	1907.	1908.
Value of plant	$380,296	$453,768
Additions to depreciable property	81,707	13,887
Increase in land value	1,620	...
Depreciation	21,281	25,482
Interest at 8%	30,424	36,301
	$515,328	$529,438
Net earnings	61,560	78,987
Value of plant at end of year	$453,768	$450,451

Under such an examination the electric plant showed a value in 1908 of $450,451, against the Commission's valuation of $534,837. The Commission held, therefore, that the plant had been earning more than the 8% assumed as a "reasonable return" and had no going value above the valuation.

CAPITALIZATION OF FRANCHISES.—Although the company did not specifically claim that the intangible values should include the franchises, yet the Commission touches on this in one paragraph:

> When it comes to capitalization of the exclusive right to perform the services that are rendered by public utilities, the situation appears to be different. Such exclusive privileges are monopolistic in their nature, due to natural conditions, and belong to the community. They have been created by the growth of population, and by economic and social development generally, rather than by any individual effort. Belonging to the public, the right of controlling and disposing of such privileges also rests with the community. Under these conditions there appear to be no grounds upon which the value of exclusive privileges of this kind should become private property. This is especially true where, as in this state, the trend of legislation is such as to reserve such values for the public.

Reasonable Return.

The decision gives careful consideration to the question of a proper rate of return to corporations supplying public service. It lays down the general rule that this is determined by the ordinary rates for capital similarly invested in other enterprises, under similar conditions, and sufficiently large to encourage investors to enter such enterprises. The company claimed consideration for such elements of risk as (1) competition with rival companies and rival forms of illuminants; (2) the danger of necessary change in methods of operation; (3) the fixed character of investment; (4) the length of time necessary to turn over the capital; (5) the necessity of continuous operation even in event of a strike or high prices; and (6) the tendency toward legislation against public-utility corporations. The Commission gave careful consideration to these elements of risk. It states:

> The greatest risks usually prevail in competitive undertakings. In these there is a constant struggle between the competitors to reduce the cost of production and to bring about other changes that will give them some advantage in the market. Such producers have no way of controlling the supply of their products and since the prices of these are, therefore, beyond their control, they are apt to suffer from any improvement in method of production on the part of any of their competitors, and this tends either to reduce the cost of these products or to enhance the demand for them in the market. In industries where there are certain monopolistic conditions such as public utilities, competitive risks are of much smaller importance. In such industries the supply is under control; there is no direct competition in the sale of the products or services. There are, of course, exceptions to this, but they are not frequent or often serious or permanent.
>
> But there are, in public utilities as in other industries, other than competitive risks. In the construction and operation of such plants many accidents may be met with and many mistakes occur. While some of these might have been foreseen and prevented, others may be beyond human intelligence and grasp. Such plants may be injured by the diversion of the growth of the city in a different direction than that expected when the plants were built, by the failure of the city to grow as rapidly as expected or as rapidly as the plant has made preparations for; by the failure of the city to grow at all as well as by decreases in its population and industries; by actions of the local and other authorities by which unprofitable extensions may be required, the return reduced or other burdens imposed, as public utilities usually have to furnish adequate service whether paying for it or not. In the case of such losses the owners or employers are the first to suffer, as their share of the proceeds is not fixed but consists of what is left after other claims have been made. Wages, salaries, supplies, taxes, interest on bonds and notes, etc., must be paid by the employers or the business will stop or go into receivership. If the earnings are only large enough to cover these outlays the employer will have to go without his pay. There is no escape from this. In view of these and other facts it is clear that public utilities are not exempted from risks, and, therefore, there is a speculative feature about them for which their owners are entitled to have something in the way of speculative gains. . . . Under the circumstances of this case it is not unreasonable to limit returns for both interest and profits to not less than from 7 to 7½% on a fair valuation on the gas plant, and to not less than 8% on a fair valuation of the electric plant.

Depreciation.

Recognition is given to the fact that some allowance should be made for depreciation and the economic necessity of such a charge is pointed out thus:

> It is admitted that, generally speaking, an operating utility plant is limited in life; even where current repairs are properly met and the utility properly maintained, there exists a loss of value directly dependent on the length of operation; such losses are always present whether a plant be in its initial or last steps of operation, and that such losses are properly borne by the consumers and properly made a charge against the revenues of the company; such an allowance is in keeping with the provisions of the Utilities Law providing that depreciation is to be considered in determining reasonable rates.
>
> It seems certain, however, that under the law it is not the purpose of the "Depreciation Reserve" to insure public utilities against all business losses in capital. Undoubtedly such losses as wear and tear, abrasion, corrosion, deterioration due to time and the elements, and other items of this character, are a cost of producing the service rendered, and must, therefore, be borne by the consumers. This would also seem to be true of

obsolescence due to progress in the art especially when the cost resulting therefrom is not offset by increased efficiency afforded either by increases in the earnings or reductions in the operating expenses. Inadequacy due to the growth of the business, particularly when it results in losses to the utility, would also seem to come in this class. Losses, on the other hand, which are due to unnecessary errors in the construction and equipment of the plant, to lack of ordinary economy, foresight and efficiency in management, would not seem to be proper charges to operating expenses. Many other losses such as those due to unforeseen competitive conditions, strikes and unexpected contingencies and accidents of various kinds, are too problematic in character to be dealt with through the Depreciation Fund alone.

From the data and tables of average life prepared by the engineering staff, it was determined that the average life of the electric plant was about 17.5 years and of the gas plant about 30.4 years. This means an annual depreciation of 3⅛% on the depreciable property of the gas plant and 5¾% on the depreciable property of the electric plant. The company claimed that at least 1% additional should be allowed in order to care for obsolescence and inadequacy and that 2% further should be added for unusual contingencies, accidents, etc. The Commission allowed straight depreciation on the electric light plant figuring an 18-year life, and computed the depreciation reserve for the gas plant on a 2% sinking-fund basis, 30-year life.

Rates and Service.

Criticism is made of the existing schedule of rates on the ground that it results in inequalities of charges between similarly situated consumers and, in instances, make it possible to purchase more current for less money. In the schedule of rates ordered effective by the Commission, the schedule of charges is based both upon the size of consumers' installation and the current consumed. The company's old schedule for light and power are based upon current consumed only. It is obvious that where changes are made in the fundamental basis of the rate and present inequalities are rectified, the present consumers will not share alike in the general reduction of the company's revenue. The great majority of consumers will, however, receive considerable reduction. For example, the proposed lighting schedule will mean reductions to all lighting consumers now paying the company's 16-ct. rate, to all lighting consumers now paying company's 14-ct. rate who use an estimated portion of their installation of 30 hours per month, to all power users now paying company's 8-ct. rate who use an estimated portion of their connected capacity 34 minutes daily, and to all power consumers now paying company's 7-ct. rate who use an estimated proportion of the connected capacity 45 minutes daily, etc.

A portion of the complaint presented related to gas and electric service and alleged a low heating and lighting value for gas due to adulteration and irregular and deficient lighting service. Periodic investigations have been made of conditions of service from time to time in both this and other cities and reports of the gas and electric inspectors making such investigations are made the basis of the ruling of the Commission on this point. The decision shows that present conditions are under most careful control and from the report of the inspections it appears that the service in general is not only satisfactory and in conformance with the rules, but that it now compares favorably with the service furnished in any other city in the state.

A MOVING SIDEWALK is likely to be installed on 34th St. in New York City. At a meeting of property owners on 34th St. on March 22, the vote favored the construction of the sidewalk by a large majority. This invention originated nearly twenty years ago. An exhibition system was operated on a pier at the Chicago Exposition in 1894, and a line nearly a mile long was operated at the Paris Exposition in 1900. No other commercial installations of the invention have ever been made, but it has been several times brought forward and on the point of adoption as a transit line in New York City. The invention is controlled by the Continuous Transit Securities Co., of which Mr. Max E. Schmidt, M. Am. Soc. C. E., 45 Broadway, New York City, is President. The proposed line on 34th St. will extend clear across the city and will be on a level about 20 ft. below the pavement.

Extracts from the Testimony of Arthur P. Davis, Chief Engineer of the U. S. Reclamation Service before the Committee to Investigate the Interior Department and Forestry Service.

Mr. Pepper [attorney, prosecuting the investigation]. Where were you, Mr. Davis, at the beginning of March, 1909?

Mr. Davis. In February, 1909, I went to Porto Rico, in accordance with the statute of the Territory of Porto Rico, requiring that the irrigation system, which they were planning, should be passed upon by the engineers of the Reclamation Service. I returned to Washington from Porto Rico on March 16.

Mr. Pepper. And did you, on the 17th of March, have an interview with Mr. Ballinger?

Mr. Davis. Secretary Ballinger telephoned me shortly after noon on the 17th of March, asking me to meet him at his hotel that evening at 7.30 o'clock, which I did. No one was present at the interview except Mr. Ballinger and myself.

Mr. Pepper. Will you state whether or not at that conversation the question of the power-site withdrawals came up for discussion; and, if so, how?

Mr. Davis. It did. The Secretary opened the conversation by saying he wanted to ask me some questions about the Reclamation Service, and proceeded to do so. He made a number of criticisms on the past conduct of the service. Among others was the fact that there had been large areas of land withdrawn for conservation of power sites which he said was illegal, and he cited a number of other things that had been practiced in the Reclamation Service which he also said were illegal.

Senator Flint. Mr. Pepper, would it interrupt you or the witness to have him fully state, not limiting it to the power sites, but all the objections that the Secretary made to what had taken place in the Reclamation Service?

Mr. Pepper. Certainly not. Mr. Davis, will you give the conversation, as nearly as you can recollect it, and if Mr. Ballinger made some general expressions as to his favorable attitude toward conservation, bring that out, too.

Mr. Davis. It has been a year now, about, and I can not make a very accurate account of the conversation, but I can tell in general the trend of it. The Secretary, as I remember it, had no commendation whatever to make of the Reclamation Service, but he criticised it on many points. One was the withdrawal for power sites; one was the fact that many settlers were on lands, who, he thought, had been promised water and had been waiting and living on promises for some time and had been misled; one was that part of our work which was going on under what we call "force accounts"—that is, the direct employment of labor—which he said was illegal and should not be done; and it should all be done by contract. Another was what he called our "publicity bureau"—that is, work, which I explained to him that evening, consisted of the dissemination of information concerning the projects, mainly to inquiring settlers, and also to the preparation of the action of the department and the current news of the service in the form of typewritten bulletins, which were hung on a hook in the office for the convenience of newspaper correspondents who might come in desiring news. I explained that that was done for the accommodation of the newspaper men and to save the time of the men who would have to talk to them, and eliminate inaccuracies which might occur in transcribing conversations instead of having them drawn in the proper way.

There is one other point that he criticised, and that was the alleged oppression of contractors. He said he had heard a great deal of complaint about the treatment of the contractors by the Reclamation Service; and in reply to that I told him that if there was anyone at fault in that it was myself, because our contracts left decisions concerning contractors to the chief engineer, and that I had made every effort I knew how to put forth to do justice to contractors by giving them full hearings and investigating as fully as possible, both personally and by testimony, all the claims that they put forth. He also criticised our having taken up too much work. Each one of these criticisms I answered in such a way as I thought proper and just, and we talked for about two hours.

Mr. Pepper. Did he seem to show a detailed familiarity with the work of the service?

Mr. Davis. No, sir. In most cases where he criticised he listened with attention to what I had to say on the subject, and seemed to be interested, and I think in nearly every case he stated that it looked much better to him; that he had not known very much about it, and he made a broad statement to that effect at the end of the evening—that he knew very little about the service, and he wanted assistance in getting acquainted with it, and that he wanted me to assist him in getting acquainted with it, which I promised to do.

Mr. Pepper. And did you know at the time whether or not he had previously taken the matter up with the director?

Mr. Davis. No, sir; I didn't. I had very little conversation with Mr. Newell, or anyone else that day, regarding the new administration, but I had asked Mr. Newell how they were coming along with the new Secretary, and he passed it off with the casual remark, "pretty well," or something like that. He said very little about it, so that I had no accurate idea of the relations at all until that evening when I got a very much better idea because the Secretary expressed a lack of confidence in Mr. Newell's ability. That was no surprise to me, because there had been a great deal of newspaper talk on that subject; in fact, while I was in Porto Rico I saw a newspaper clipping that was sent down there, dated in February, which announced that the Secretary of the Interior was going to make changes in the Reclamation Service, among which was the removal of the director.

Mr. Pepper. Mr. Davis, in the course of Mr. Pinchot's letter to the President of Nov. 4, 1909, this language appears:

Under Mr. F. H. Newell, as director, the United States Reclamation Service has become an organization of exceptional efficiency. It contains many engineers of high character and standing, who are engaged on its projects at a fraction of what they could earn elsewhere. During the spring I became greatly concerned because it was coming to be generally believed in the Reclamation Service that the director had lost the support of Secretary Ballinger. Some of the best engineers were resigning and others were considering the same step. The situation demanded prompt action. Accordingly, in the hope of preventing further loss to one of the most important branches of practical conservation, I laid the matter before you. You assured me that the Reclamation Service would be protected. No one can doubt your own purpose in this matter. Yet I recognize with regret that the unfortunate situation to which I called your attention still exists. If it is allowed to continue, the Reclamation Service must inevitably disintegrate.

Leaving out the question of Mr. Pinchot's interview with the President, which I assume you know nothing of, is the statement that Mr. Pinchot makes, in the letter I just read to you, a substantially correct statement as of its date, Nov. 4?

Mr. Davis. In my opinion it is.

Mr. Pepper. Is it a substantially correct statement as of the present time?

Mr. Davis. In my opinion it is.

Mr. Pepper. In Mr. Ballinger's letter to the President dated Nov. 15 he says:

The statement that the Reclamation Service had lost the support of the Secretary is without foundation, as is also the further statement that it is in danger of disintegration through any act of commission or omission on my part. The law lays upon the head of the Interior Department great responsibilities in its administration in reclamation matters. To say that efforts to become familiar with the operations of the service and to intelligently carry out the duties incident to that administration will accomplish disintegration, is entirely unwarranted and involves the assumption that the conduct of the reclamation officers can not stand scrutiny or supervision. As none of said officers have ever intimated any such idea to me, and it is unfair to assume that they are so regardless of their obligations to their responsible head as to secretly indulge in criticism to an officer who has no legal or other right to interfere in their affairs, it is reasonable to believe that the criticism made is based on idle gossip, to which no unprejudiced person, devoid of anxiety to condemn without a hearing, would have paid any heed.

Mr. Pepper. Mr. Davis, what is the fact as to what has passed between you and Secretary Ballinger in regard to the Reclamation Service at any interview which you are now able to specifically recall?

Mr. Davis. Before the 4th of March, 1909, frequent newspaper items appeared which announced that Secretary Ballinger was intending to make drastic or radical changes in the Reclamation Service, including the removal of the director. When I returned to Washington, in conversation with various individuals, it was told to me as coming with some directness, or comparative directness, from the Secretary's office that he was intending to make radical changes. Similar statements were made to me by the Secretary himself, I think beginning on March 17, in our first interview.

On the 10th day of June Secretary Ballinger, in his office, stated to me that he had definitely decided upon the long-expected movement of changing the head of the Reclamation Service. As he was telling me that in an official capacity I had no doubt it would soon be carried out, and told him that I believed my duty was to state to him what I believed would be the result. I told him that so far as my knowledge went that the officers of the Reclamation Service had confidence in the present director, and I believed his removal under these circumstances, before the Secretary had investigated the service, would be taken as an announcement by the service that merit did not count under this administration. I told him that undoubtedly it would be possible to live down that impression if nothing further occurred, but I believed that that would be the general impression; that, however, was only an opinion, of course, and he of course so understood it.

He disclaimed any desire to do any injustice to Mr. Newell, and began to discuss the possibility of other positions in which to place him, whether as consulting engineer in the Reclamation Service or various other

capacities in the government employ, and I replied that I thought the matter was comparatively insignificant when compared with the interests of the Reclamation Service as a whole, and the integrity of the work. I told him then—in as strong language as I felt politeness and the proper respect would permit—that in my judgment his entire course since he had been announced as Secretary of the Interior, so far as my knowledge went, had been one that was subversive of the interests of efficiency in the Reclamation Service and tended to its disintegration. He asked me what I referred to, and I replied that I referred to the persistent gossip and rumor that these drastic changes were to be made. He protested that these announcements in the press were unauthorized, in which I, of course, acquiesced, but reminded him that perhaps, although unauthorized, they were nevertheless true in accordance with his frequent statements to me, and furthermore, that they had appeared with such circumstantial coordination, and we had verbal assurances from the newspaper men who wrote some of them that they were substantially the words of the Secretary himself, and that at any rate the public and the service regarded them as semi-authorized, at least, and as accurately representing the Secretary's views. He disclaimed any intention to injure the service, and I think that was about all that happened at that interview. I did express in as emphatic terms as I could, the sentiments I have now related.

Mr. Pepper. Now, Mr. Davis, following upon this conversation on the 10th of June, can you tell us what has actually happened in the relations between the Secretary and the Reclamation Service? Has there been any action taken by him in connection with the affairs of the service that is germane to that conversation?

Mr. Davis. One week after that, the 17th of June, appeared a long item in the Washington "Post," and near the same date a column or two in the Boise (Idaho) "Statesman," which presented substantially the same sentiments which he had expressed to me of his decision to remove the Director of the Reclamation Service and to make other changes. From time to time he made statements to me that our salary grade was too high, and that he proposed to make radical changes in the service. Those statements were frequently made, and the last time I believe they were made was a week ago yesterday, or reiterated. The Secretary told me he proposed to reorganize the Reclamation Service and that our scale of salaries was too high. Some time, I think in May, perhaps, one of our engineers who was receiving a salary of $3,300 was offered $4,800 to go into other work, and he came to me.

The Chairman. Who was that?

Mr. Davis. Mr. C. H. Fitch. He came to me and said that he had adopted the reclamation work as his life work, and he felt very much attached to it, and desired to remain, but he did not feel that he could overlook an offer of this kind without finding out what his status was and whether his prospects in the Reclamation Service were good. I told him then that, while I did not care to advertise the fact, I thought it my duty in justice to him to tell him the Secretary's sentiment toward the people of the Reclamation Service; that I did not know whether or not he would be reduced in salary or dismissed, or changed in anyway, but I told him, as near as I could remember them, the facts of the Secretary's attitude toward the service, and that I could not predict the future at all. Mr. Fitch left the service, and is now in the employ of a company.

Mr. Pepper. Mr. Davis, is it not a fact that there has been a direct intervention with the Secretary of the Interior in the relation between the Reclamation Service and the contractors during the last year?

Mr. Davis. There was one case. In our specifications for large and important concrete structures we have a proviso allowing the engineer to stop the work where, in his judgment, climatic conditions require it. In the case of high dams, we are expected to do that in the summer, owing to the inferior qualities of the concrete placed in extremely hot dry weather. There are three reasons for that. One is that where a long and not very thick wall occurs, if it is built in hot weather it contracts when cold weather comes, and is likely to cause cracks which, of course, are undesirable in dams. Another reason is that it is difficult to keep the cement sufficiently worked during the period when it should be hydrated, and it may not fully hydrate and get its full strength; and in the southern country, where it is exceedingly hot in the summer, there is another very serious difficulty, and that is that the temperature causes the cement to set prematurely. It sets sometimes so quickly that it can not be used, and sometimes it has to be rejected when it will pass all the tests well in a temperate climate, such as San Francisco.

That is regarded as a very important matter in the Reclamation Service, and we provide in our specifications for controlling the character of the work. The reservoir dam in southern Arizona is approaching completion and had reached a high point last summer. The engineer directed the contractor to stop work at a certain time, which he designated, and he promised to do it, which is, in my judgment, clearly provided in the contract and was always so intended when the contract was written, at least; and the contractor, not caring to have his force interfered with, thought he would get that changed. He appealed to the Secretary of the Interior, and, without referring the matter to the Reclamation Service or finding out the reasons for the order, the order was reversed by the Secretary's office.

Mr. Pepper. What, if anything, did you do with that matter?

Mr. Davis. I brought the matter to the Secretary's attention. I was traveling in the West, and the Secretary had started west just about the time the order was given. The order I believe was given by the Assistant Secretary, but the matter was considered by the Secretary himself here in Washington. On receiving notice to the effect that the engineer was reversed, the director took the matter up with the Assistant Secretary, who said that he could not change the matter; it had been decided by Secretary Ballinger himself. Secretary Ballinger was in the West, and on the 14th day of July, I think it was, I met him in Seattle and laid the matter before him, and explained to him the reasons for the order. We had also had considerable telegraphic correspondence with the engineer, in which he had been endeavoring to reach a compromise with the contractor in such a way as to cause the least inconvenience and do the least damage to the work. It was such a compromise as would naturally have followed the order, and that compromise was finally approved by the Secretary, by which the contractor could carry on certain work which was not so detrimental at that time, and it was arranged without damage to the United States and to the mutual satisfaction all around. But that character of action of course loosened the authority of the engineer over the contractor. It tends to put the engineer in contempt of the contractor and to instill in the engineers a fear of carrying out what is their plain duty, in my judgment.

Mr. Pepper. A statement was made to the effect that there had been a notable or considerable number of resignations from the Reclamation Service during the past year. Is that a mistake or is it a fact?

Mr. Davis. It is a fact. There has been a number of them, but not very much greater than the normal, perhaps. The greatest difference is in the importance and the causes. There are a number of engineers, in fact most of our prominent engineers, who are occasionally receiving opportunities for employment from the outside. Their chief tie to the service is not the matter of salary, but interest in their work and loyalty to it, and the belief that they are appreciated, and that there is a future for them in the service. For instance, one of the supervising engineers who was receiving a salary, I believe, of $4,500 per annum from the Reclamation Service was offered double the salary by a large contracting firm, with also an interest in the profits. He did not accept the offer. He has had a similar offer since and has turned it down.

The Chairman. What engineer was that?

Mr. Davis. Mr. Louis C. Hill. Another engineer, Mr. I. W. McConnell, who was receiving a salary of $4,800 per annum was, two years or so ago, offered $8,000 by a firm in the line of business in which he was engaged, and he refused the offer. The offer was repeated about six months ago and he accepted it, and from his sentiments I think it was very largely due to the feeling that there was a change in the policy of the service.

There has been the repeated announcement, not publicly, but that the engineers knew about, that the salary list is too high, which is practically an announcement that there is no future for them in the way of salary, and the policy of withholding confidence from the engineers and reducing the confidence that is reposed in them also has a similar effect.

There are a large number of promotions that have been made in the interest of the service that have been held up a long time. In fact, there have been in the last six months always pending in the Secretary's office for a good many weeks at a time a large number of promotions that have been recommended by the supervising engineer, and indorsed by our office, but not approved by the Secretary. Very few have been turned down, but the postponement has the moral effect of convincing the men that even if they get promotions that they come grudgingly and they cannot expect anything. We have had a number of resignations of men in lower positions, and some of them have taken the trouble to state in their resignations that it was due to the fact that the conditions were unsatisfactory in the service, and that same sentiment has been expressed by others who are still in the service.

I have had men in the service ask me to pass the word to them so that they could resign in a body, as a protest against the handling of this service by the present Secretary. I have always discouraged that attitude and told them that that was not the attitude of true loyalty to the service, and have never encouraged it in any way; but some of the engineers feel very strongly on that subject. I had quite a conversation with Secretary Ballinger a week ago yesterday on these various points that I have spoken about, and yesterday I received this telegram from a supervising engineer:

Huntley, Mont., March 10, 1910.

Director Reclamation,
Washington, D. C.:

L. W. Stockman, engineer, and Otto Brackman, bookkeeper, resigned to accept materially better paying positions, crippling organization. Immediate confirmation recommended; promotions urgent.

Savage.

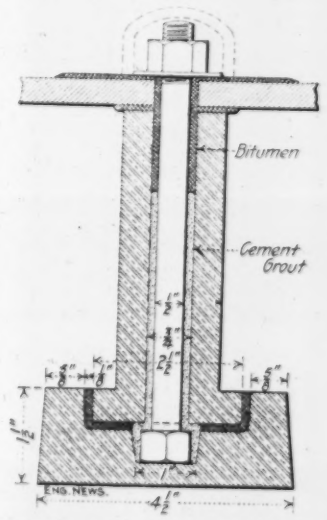
It appears that those promotions have been approved; they were approved very promptly, subsequent to my interview last 4th of March with the Secretary, but there are still a large number held up. The Secretary gives as a reason for this that he intends to reorganize the service. He gave me that reason a week ago yesterday, and that reason alone, aside from the matter of promotion, is one that spreads this same impression.

Now, an organization like the Reclamation Service, with a large number of highly trained men, of necessity is a very difficult thing to build up to a state of efficiency, and the loss of a single man like I. W. McConnell, who resigned last summer, is, I am convinced, for the reason that I have stated, absolutely irreparable. If we can find another man like McConnell, we will have one instead of two, as we ought to have. It has been the constant effort and the chief function of the Director and myself to find the right kind of men; to recognize ability when we see it, and keep it satisfied with appreciation so that we can hold it. The highest function of administrative ability is that of recognizing ability in others and commanding it. The Reclamation Service does not pay high enough salaries to hold men by the inducement of the salary alone, and in my judgment it is unnecessary.

Past experience has shown that it is not necessary to pay as high salaries in order to command the same ability as is paid in the commercial world, but it becomes necessary if you deprive the men of the inducement to loyalty and pride in the service and the satisfaction that they get from the respect and approval of their superiors, and when that is withheld it removes an immense money consideration. I am convinced that the efficiency of the majority of the men who have remained in the service is very greatly decreased by the depression that has been produced by the unsympathetic attitude of the present Secretary.

A Protection Against Electrolysis for Anchor Bolts.

The corrosion of anchor bolts for structures known to carry electric currents have often been noted. Perhaps the most notable instances reported have been of bolts at the base of columns supporting the elevated-railroad structures in New York City. It was pointed out in Engineering News, Dec. 24, 1908, p. 719, that there was a possibility that, in course of time, the foundations of steel-frame buildings might become corroded by currents, passed into the steel



An Insulator for Protecting Anchor Bolts Against Electrolysis.
(Patented by Mr. R. T. Kanski, New York City.)

frame, from pipes which acted as carriers of currents picked up from an existing network and delivered into areas of marked relative negative potential.

In a letter to the editor, printed in Engineering News, Aug. 20, 1908, Mr. R. T. Kanski, of New York City, showed a vitrified-clay protector for anchor bolts. By this device it was claimed that holding-down bolts of all sizes could be completely insulated without any decrease in the load imposed on each bolt as embedded in foun-

dations. This device has been patented and is being marketed, in the form shown in the accompanying figure, by the Anti-Electrolytic Co., Inc., 20 Broad St., New York City.

To secure authentic data on the ability of the vitrified-clay protectors to remain intact in service a series of tests were made on two sizes by Prof. Ira H. Woolsen, at the Columbia University laboratories in New York City.

With 1-in. bolt and proper insulator, embedded in concrete, the bolt broke at the thread with a load of 27,700 lbs. The insulator showed no signs of failure. With a 1.5-in. bolt and proper insulator, the concrete cracked at 43,900 lbs., and the bolt broke in the middle with a load of 86,320 lbs. The insulator showed no signs of damage.

The foregoing tests were repeated, but the insulators were not embedded in concrete. In the test of a 1-in. bolt, the flange of the insulator cracked at 16,000 lbs., and at 29,400 lbs. the bolt broke in the middle. While the flange of the insulator was broken, the shank was uninjured. With an insulator, for 1.5-in. bolts, three bolts were used. The first broke with 46,200 lbs. load at a defective weld. The second bolt broke at 51,530 lbs., but the insulator was uninjured. The third withstood a load of 54,000 lbs., at which the insulator cracked.

From these tests it appears that a vitrified insulator of this type can be secured which is as strong as desired without being exceedingly cumbersome.

The general proportions of the common insulator are indicated in the accompanying figure, though the dimensions are for the ½-in. bolt size only. The waterproof joints, introduced to give a certain range of adjustment of bolt and insulator, are made with a filling of any desirable bitumen compound. Under the bolt head an impregnated paper may also be used to give more body to the filling. If it is desired to cover the nut and end of the bolt with earth or concrete a hemispherical vitrified cup is sealed on top.

As the potentials found between various buried structures are known to be of the order of 10 or 15 volts, the insulation provided by the vitrified clay and bitumen joints is ordinarily sufficient.

Rail Specifications and Sections*

SPECIFICATIONS FOR RAILS.—A new specification should not be prepared at this time without careful consideration. So far as we know, no railway has purchased rails under the specifications approved by the American Railway Association. Nor do we know of any railway company that has succeeded in buying rails during the past two years according to a specification entirely satisfactory to the railway company. We believe that all of the specifications under which rails have been rolled have been compromises on the part of both parties, with the general result that neither party is entirely satisfied. Our experience during the year has brought to our attention some defects in all of the specifications now before us, and acting under the impression that there is a distinct feeling that we should revise our specifications, we offer the attached specifications for consideration.

We believe it necessary to submit a sliding scale for the percentages of carbon and phosphorus, which provides for increasing the carbon as the phosphorus decreases. The fixing of this scale properly is a matter requiring care and we admit that our knowledge on the subject is limited. The American Railway Association specification calls attention to this matter in the following words: "When lower phosphorus can be secured, a proper proportionate increase in carbon should be made." The amount of increase is not provided for in the specifications and this appears to us to be necessary in order to secure uniformity of practice; otherwise, the fixing of these percentages becomes a matter of special arrangement.

Bessemer rails are being furnished regularly with phosphorus under the maximum allowed, and where this is done, the carbon should be raised above the higher limit now fixed in our specifications, or a soft and poor wearing rail will result; yet this condition has not been fully guarded against in rails furnished under existing specifications. The lower and upper limits for carbon have heretofore been fixed with the intention that the mills furnish rails with a composition as near between the two limits as possible. The mills, however, in order to meet the prescribed drop tests with the least difficulty, keep both the carbon and manganese as nearly as possible to the lower limits, with the corresponding result that a generally poor-wearing rail is furnished.

Some roads have prescribed the limits of deflection to be allowed under the drop test. With our present knowledge, we believe that we should fix a minimum deflection to eliminate brittle rails and to secure greater uniformity of product; also maximum deflection to eliminate soft rails. We are not able at the present time to fix such limits for the specifications.

With reference to the amount of discard, time of holding in ladle, size of nozzles, and other such details of manufacture or machinery, we are of the opinion that the physical and chemical tests required should be prescribed and that we should see that the material submitted for acceptance meets the prescribed tests. We should not dictate to the manufacturers the amount of crop which shall be removed from the top of the ingot, as this should vary with the care and time consumed at the various mills. The railways should not be asked to take anything but sound material in their rails. The mills can furnish such sound material if the proper care and sufficient time are taken in the making of the ingots. Information derived from the tests being made at the Watertown Arsenal shows definitely that sound rails cannot be made from unsound ingots, and that, therefore, the prime requisite in securing a sound rail is to first secure the sound ingot.

RAIL SECTIONS.—The instructions of the American Railway Association require us to study its sections "A" and "B" in use,* and submit a single type for standard. Owing to the conditions existing in 1908, very little rail was laid, and practically none of the A. R. A. sections, in such manner as to give the needed information. This year, several roads have laid A. R. A. sections, with a view of determining their relative merits. These rails have been in the track so short a time that we are not justified in drawing any conclusions as to which of the A. R. A. types, "A" or "B" (if either), is better than the Am. Soc. C. E. sections.

The results so far obtained from the heavy base A. R. A. sections are disappointing, as we have received from the mills some rail of the new section which was as bad as what we had with the Am. Soc. C. E. section, showing that the quality of the rail does not depend entirely upon the section.

Specifications for Steel Rails.

(1) The entire process of manufacture shall be in accordance with the best current state of the art.

(a) Ingots shall be kept in a vertical position until rolled, or until the metal in the interior has had time to solidify.

(b) Bled ingots shall not be used.

(2) The chemical composition of the steel from which the rails are rolled shall be within the following limits:

	Bessemer.		Open Hearth.	
	70 lbs. to 85 lbs.	85 to 100 lbs., inclusive.	70 lbs. to 85 lbs.	85 to 100 lbs., inclusive.
Carbon	0.40 to 0.50	0.45 to 0.55	0.53 to 0.66	0.63 to 0.76
Manganese	0.80 to 1.10	0.80 to 1.10	0.70 to 1.00	0.70 to 1.00
Silicon	0.07 to 0.20	0.07 to 0.20	0.07 to 0.20	0.07 to 0.20
Phosphorus not to exceed	0.10	0.10	0.04	0.04
Sulphur not to exceed	0.075	0.075	0.06	0.06

(3) When the average phosphorus content of the ingot metal used in the Bessemer process at any mill is below 0.08 and in the open-hearth process is below 0.03, the carbon shall be increased at the rate of 0.035 for each 0.01 that the phosphorus content of the ingot metal used averages below 0.08 for Bessemer steel, or 0.03 for open-hearth steel.

The percentage of carbon in an entire order of rails shall average as high as the mean percentage between the upper and lower limits.

(4) The end of the bloom formed from the top of the ingot shall be sheared until the entire face shows sound metal. All metal from the top of the ingot, whether made from the bloom or the rail, is the top discard.

(5) The number of passes and speed of train shall be so regulated that, on leaving the rolls at the final pass, the temperature of the rails will not exceed that which requires a shrinkage allowance at the hot saws, for a 33-ft. rail of 100 lbs. section, of 6½ ins. for thick-base sections and 6¾ ins. for Am. Soc. C. E. sections, and ¼-in. less for each 10 lbs. decrease of section, these allowances to be decreased at the rate of 1-100-in. for each second of time elapsed between the rail leaving the finishing rolls and being sawed.

The bars shall not be held for the purpose of reducing their temparature, nor shall any artificial means of cooling them be used between the leading and finishing passes, nor after they leave the finishing pass.

(6) The section of rail shall conform as accurately as possible to the templet furnished by the railway company. A variation in height of 1-64-in. less or 1-32-in. greater than the specified height, and 1-16-in. in width of flange, will be permitted; but no variations shall be allowed in the dimensions affecting the fit of splice bars.

(7) The weight of the rail shall be maintained as nearly as possible, after complying with the preceding paragraph, to that specified in the contract.

A variation of 0.5% from the calculated weight of section, as applied to an entire order, will be allowed. Rail will be accepted and paid for according to actual weight.

(8) The standard length of rail shall be 33 ft.; 10% of the entire order will be accepted in shorter lengths varying by 1 ft. from 32 ft. to 25 ft.

A variation of ¼-in. from the specified lengths will be allowed. All No. 1 rails less than 33 ft. shall be painted green on both ends.

(9) Care shall be taken in hot-straightening rails, and it shall result in their being left in such condition that they will not vary throughout their entire length more than 4 ins. from a straight line in any direction for thick base sections, and 5 ins. for Am. Soc. C. E. sections when delivered to the cold-straightening presses. Those which vary beyond the amount, or have short kinks, shall be classed as second quality rails and be so marked.

The distance between supports of rails in the straightening press shall be not less than 42 ins.; supports to have flat surfaces and out of wind. Rails shall be straight in line and surface and smooth on head when finished, final straightening being done while cold.

They shall be sawed square at ends, variations to be not more than 1-32-in., and prior to shipment shall have the burr caused by the saw cutting removed and the ends made clean.

(10) Circular holes for joint bolts shall be drilled in accordance with specifications of the purchaser. They shall in every respect conform accurately to drawing and dimensions furnished and shall be free from burrs.

(11) The name of the manufacturer, the weight of the rail, and the month and year of manufacture shall be rolled in raised letters and figures on the side of the web. The number of the heat and a letter indicating the portion of the ingot from which the rail was made shall be plainly stamped on the web of each rail, where it will not be covered by the splice bars. Rails to be lettered consecutively A, B, C, etc., the rail from the top of the ingot being A. In case of a top discard of 20% or more the letter A will be omitted. Open-hearth rails to be branded or stamped O. H. All marking of rails shall be done so effectively that the marks may be read as long as the rails are in service.

(12)* Drop tests shall be made on pieces of rail rolled from the top of the ingot, not less than 4 ft. and not more than 6 ft. long, from each heat of steel. These test pieces shall be cut from the rail bar next to either end of the top rail, as selected by the inspector.

The temperature of the test pieces shall be between 40° and 100° F. The test pieces shall be placed head upward on solid supports, 5 ins. top radius, 3 ft. between centers, and subjected to impact tests, the tup falling free from the following heights: 16 ft. for 70-lb. rail; 18 ft. for 80, 85 and 90-lb. rail, and 20 ft. for 100-lb. rail.

The test pieces which do not break under the first drop shall be nicked and tested to destruction.

(13a) Two pieces shall be tested from each heat of steel. If either of these test pieces breaks, a third piece shall be tested. If two of the test pieces break without showing physical defect, all rails of the heat will be rejected absolutely. If two of the test pieces do not break, all rails of the heat will be accepted as No. 1 or No. 2 classification, (according as the deflection is less or more, respectively, than the prescribed limit).†

(b) If, however, any test piece broken under test (a) shows physical defect, the top rail from each ingot of that heat shall be rejected.

(c) Additional tests shall then be made of test pieces selected by the Inspector from the top end of any second rails of the same heat. If two out of three of these second test pieces break, the remainder of the rails of the heat will also be rejected. If two out of three of these second test pieces do not break, the remainder of the rails of the heat will be accepted, provided they conform to the other requirements of these specifications, as No. 1 or No. 2 classification, (according as the deflection is less or more, respectively, than the prescribed limit.)

(d) If any test piece (test a) does not break, but when nicked and tested to destruction shows interior defect, the top rails from each ingot of that heat shall be rejected.

(14) The drop-testing machine shall be the standard

*Abstract of the report of the Rail Committee presented at the annual meeting of the American Railway Engineering and Maintenance-of-Way Association, held at Chicago, on March 15, 17.

*Engineering News, May 14, 1908, and March 11, 1909.

*It is proposed to prescribe, under this paragraph, the requirements in regard to deflection, fixing maximum and minimum limits, as soon as proper deflection limits have been decided upon.

†The clause in brackets in sections (a) and (c) of paragraph 13 to be added to the specifications when the deflection limits are specified.

of the American Railway Engineering and Maintenance of Way Association, and have a tup of 2,000 lbs. weight, the striking face of which shall have a radius of 5 ins. The anvil block shall be adequately supported and shall weigh 200,000 lbs. The supports shall be a part of or firmly secured to the anvil.

(15) No. 1 rails shall be free from injurious defects and flaws of all kinds.

(16) Rails which, by reason of surface imperfections, are not accepted as No. 1 rails, will be classed as No. 2 rails, but rails containing physical defects which impair their strength, shall be rejected.

No. 2 rails to the extent of 5% of the whole order will be received. All rails accepted as No. 2 rails shall have the ends painted white, and shall have two prick punch marks on the side of the web near the heat number near the end of the rail, so placed as not to be covered by the splice bars.

Rails improperly drilled, straightened, or from which the burrs have not been properly removed, shall be rejected, but may be accepted after being properly finished.

Different classes of rails shall be kept separate in shipment. All rails shall be loaded in the presence of the inspector.

(17a) Inspectors representing the purchaser shall have free entry to the works of the manufacturer at all times while the contract is being executed, and shall have all reasonable facilities afforded them by the manufacturer to satisfy them that the rails have been made in accordance with the terms of the specifications.

(b) For Bessemer steel the manufacturer shall, before the rails are shipped, furnish the inspector daily with carbon determinations for each heat, and two complete chemical analyses every 24 hours representing the average of the other elements (specified in section 2) contained in the steel, for each day and night turn respectively. These analyses shall be made on drillings taken from the ladle test ingot not less than ¼-in. beneath the surface.

For open-hearth steel, the makers shall furnish the inspectors with a complete chemical analysis of the elements specified in section 2 hereof for each melt.

(c) On request of the inspector, the manufacturer shall furnish drillings from the test ingot for check analysis.

(d) All tests and inspections shall be made at the place of manufacture, prior to shipment, and shall be so conducted as not to unnecessarily interfere with the operation of the mill.

Impact Tests on Bridges, and the Effect of Flat Spots on Car Wheels.*

IMPACT TESTS.—(1) With track in good condition the chief cause of impact was found to be the unbalanced drivers of the locomotive. Such inequalities of track as existed on the structures tested were of little influence on impact on girder flanges and main truss members of spans exceeding 60 to 75 ft. in length.

(2) When the rate of rotation of the locomotive drivers corresponds to the rate of vibration of the loaded structure, cumulative vibration is caused, which is the principal factor in producing impact in long spans. The speed of the train which produces this cumulative vibration is called the "critical speed." A speed in excess of the critical speed, as well as a speed below the critical speed, will cause vibrations of less amplitude than those caused at or near the critical speed.

(3) The longer the span length, the slower is the critical speed and, therefore, the maximum impact on long spans will occur at slower speeds than on short spans.

(4) For short spans, such that the critical speed is not reached by the moving train, the impact percentage tends to be constant so far as the effect of the counterbalance is concerned, but the effect of rough track and wheels becomes of greater importance for such spans.

(5) The impact as determined by extensometer measurements on flanges and chord members of trusses is somewhat greater than the percentages determined from measurements of deflection, but both values follow the same general law.

(6) The maximum impact on web members (excepting hip verticals) occurs under the same conditions which cause maximum impact on chord members, and the percentages of impact for the two classes of members are practically the same.

(7) The impact on stringers is about the same as on plate girder spans of the same length and the impact on floor beams and hip verticals is about the same as on plate girders of a span length equal to two panels.

(8) The maximum impact percentage as determined by these tests is closely given by the formula:

[Editor's Note: At the hour of going to press the formula as given in the committee's printed report is found to be erroneous and we therefore omit it. The correct formula will be given in our next issue.]

(9) The effect of differences of design was most noticeable with respect to differences in the bridge floors. An elastic floor, such as furnished by long ties supported on widely spaced stringers or a ballasted floor, gave smoother curves than were obtained with more rigid floors. The results clearly indicated a cushioning effect with respect to impact due to open joints, rough wheels and similar causes. This cushioning effect was noticed on stringers, floor beams, hip verticals and short-span girders.

(10) The effect of design upon impact percentage for main truss members was not sufficiently marked to enable conclusions to be drawn. The impact percentage here considered refers to variations in the axial stresses in the members, and does not relate to vibrations of members themselves.

(11) The impact due to the rapid application of a load, assuming smooth track and balanced loads, is found to be, from both theoretical and experimental grounds, of no practical importance.

(12) The impact caused by balanced compound and electric locomotives was very small and the vibrations caused under the loads were not cumulative.

(13) The effect of rough and flat wheels was distinctly noticeable on floor beams, but not on truss members. Large impact was, however, caused in several cases by heavily loaded freight cars moving at high speeds.

FLAT SPOTS ON CAR WHEELS.—The observations of the effect of flat spots on car wheels on bridge stresses indicate a jarring or vibratory effect rather than a material increase in the unit stresses of the several bridge members. Such a result was to be expected, but the possession of tangible evidence of this effect is a material addition to our knowledge of the behavior of bridges.

In the general discussion concerning the effect of flat wheels, it has been asserted that under a rapidly moving train the fall of a car wheel due to a flat spot is so trifling that no serious blow could be delivered. For this reason the sub-committee has sought to procure some evidence of the magnitude of the blow from a flat wheel, and through the courtesy of Mr. A. W. Johnston, General Manager of the New York, Chicago & St. Louis Ry., an attempt has been made to measure the force of the blow under working conditions.

For this purpose an 80,000-lb. capacity car was equipped with registering devices to measure the compression of the car springs and a pair of wheels with flat spots was placed in one of the trucks. The trucks were of the arch-bar type and in the center of each nest of springs there was placed a small tube in which was inserted a steel pin fitted sufficiently tight to hold it in place, but capable of being forced upward when, through the deflection of the springs, it came in contact with a projection arranged at the bottom of the nest.

The car was carefully loaded with splice bars to its full capacity, the load being uniformly distributed throughout the car. The test was made on a piece of straight track in excellent condition between Cleveland and Lorain. At the beginning of the test the steel pins were driven down in the tubes until they came in contact with the aforementioned projections. The train was then run for a short distance, brought to a stop, and the displacement of the steel pin carefully measured with calipers. After this measurement, the pin was again driven down and the process repeated. In this manner seven different tests were made. It is assumed that as the springs and recording devices are located midway between the two pairs of wheels belonging to a single track, if the wheels of one pair have flat spots, only half of the influence of those spots is recorded. This may not be precisely correct, but it seems the most reasonable assumption that can be made.

The deflection of the car springs is occasioned by many causes. First in importance is, of course, the load of the car. In addition to this every inequality of the track produces a swaying or lurching movement of the car body, which causes the springs to oscillate back and forth. Bad rails, particles of earth and stone on the rails, the starting and stopping of the locomotive, the change of speed of the locomotive, and the application of brakes all cause variations in the deflection of the springs. In order to avoid these effects as far as possible, the recording device was arranged so that it could be set at any given time and indications secured at any time thereafter, and a very smooth and straight piece of track was selected on which to make the tests. The speed record was secured by counting the joints as the train passed over them. The wheels having flat spots were placed in a lathe and found to be elsewhere perfectly circular and truly centered on the axles.

From a study of the tables of measurements it is apparent that the springs in the truck having flat wheels show a greater deflection than the springs in the other truck where the wheels had no flat spots; also that the increase of pressure on the rail due to a flat spot under the conditions of the test may amount to as much as 10,600 lbs. The pressures given are of course the number of pounds required to compress the spring the amounts mentioned. But it should be remembered that the impacts producing these deflections were made almost instantaneously and the effect is that of a blow and not a gradually applied load. This difference is very important, since the steel of which rails are made is injuriously affected by blows.

The tests appear to be a sufficient demonstration that a flat wheel does actually deliver a very serious blow on the rail, and while these tests may not be sufficient to change the opinions of those who hold that the present M. C. B. limit for flat spots of 2½ ins. is sufficient for 100,000-lb. capacity cars, they certainly indicate an abuse of the track structure which is hard to excuse and which is of serious moment.

During the progress of the special impact tests the sub-committee on impact made such observations as it could relative to the effect of flat spots on car wheels on the stresses in bridges. In one series of tests a car having a wheel with a very decided flat spot was purposely used, and in other tests wheels with flat spots were occasionally noted in the test trains. In one case the tires of the locomotive were badly worn and quite rough.

From these observations the committee is unable to report any quantitative results. A distinct effect was noticeable on such members as stringers and floor beams. Where the track was smooth, and the wheels also, the records from the instruments attached to such members were generally clear and readable, even though the stresses under high speed might be largely in excess of those at low speed. In the case, however, of a rough track, caused by an open rail joint, or a flat or rough wheel, the passing train would produce such a "jarring" action on the stringer or beam that the measuring apparatus would be so shaken as to cause the parts to vibrate excessively, producing large and rapid vibrations in the record, thus rendering the diagram unreliable and, in many cases, quite worthless.

Such diagrams would also commonly result where, for any reason, there was little or no elasticity between the rail and the beam. In one case where the rails were placed in steel troughs riveted directly to small cross-beams, the hip verticals were affected in this way. This effect would, however, seldom reach beyond the floor beams, and in the case of widely spaced stringers, would not always show up in the stringers. It would seem, therefore, that the "jarring" effect might not imply a very great increase in stress, but that there is some increase is undoubtedly true.

The effect of flat spots on main truss members of a bridge was not definitely noticed in any case, but there were several cases where a train of three or four heavily loaded freight cars in a test train caused vibrations of the whole structure quite as large, relatively to the static deflection, as those caused by the locomotive. This effect was not noted in connection with any decided flat spots in the wheels, but in some cases, at least, it seemed to be caused by a vibratory motion of a car body due to some other cause. This vibratory motion appeared to be due rather to some eccentricity of the bearing or of the wheel than to any flat spots, although our observations were not extensive enough to enable us to make a definite statement on this point. The speed at which such truss vibrations occurred under the loaded freight cars was generally high, such as 50 to 60 miles per hour. No such vibrations were ever noticed under the cars of a passenger train. The difference in equipment was very evident in the character of the diagrams produced.

*Abstracts of reports of two sub-committees of the Committee on Iron and Steel Structures, presented at the annual meeting of the American Railway Engineering and Maintenance of Way Association, held at Chicago March 15,17.

A Break in a Water Main at Springfield, Mass.

By ELBERT E. LOCHRIDGE,* Assoc. M. Am. Soc. C. E.

An accident resulting in rather serious consequences occurred at Springfield, Mass., on Monday morning, March 21, 1910, at about 9 o'clock. A break in a water main 14 ft. below the surface made such a large opening in the system that the pressure on the lower levels of the city was dropped from 140 lbs. to 25 lbs. for a period of 25 minutes. This loss of pressure entirely drained two-thirds of the piping mileage within the city, thus entirely cutting out the hill section from water. Instead, however, of the 36 hours required to restore conditions of service which was necessary when a similar damage occurred in December, the mains were filled from the new Little River system on the closing of the valves near the accident within from five to ten minutes.

Water St., which is the street parallel to and lying between Main St. and the Connecticut River, was chosen as the street for the 36-in. distributing main, and at State St. a 24-in. connection was laid for a distance of 40 ft. On account of a main sewer, large electric light and telephone conduits and main gas pipes at State and Water Streets it was necessary to lay the pipe at this point at a depth of 14 ft. and this 24-in. side connection was placed from a tee below

*Chief Engineer, Water Department, Springfield, Mass.

these structures at the same time, to a point where in the future it could be connected across to Main St. with the larger size. The street at present, however, has 8 and 6-in. pipes through this one block for services and at a distance of 40 ft. from the 36-in. main a 24 × 8-in. (in two pieces) reducer was placed, directly connected to the Y from which with 90° bends the pipe rose vertically 8 ft. to where two other 90° bends connected the 6 and 8-in. pipe at the ordinary depth of 6 ft. below the surface. Each of these elbows was surrounded and backed by concrete with an amount calculated at the time to withstand all of the unbalanced pressure resulting from the elbows and reducer.

These pipes have been in this service since 1907 under pressure ranging up to about 125 lbs. up to the latter part of 1909, when the Little River pressure of 140 lbs. was admitted to the system.

The break was merely the forcing off of the reducer at the depth of 14 ft. This reducer was not found in the course of repairs, while all of the other elbows with their masses of concrete about them remained in place, although the earth was entirely removed from about them. This movement may have been due either to the fact that the concrete was not actually placed around this lower joint or that some leakage from the pipe or sewer or leakage along some other pipe or conduit made of the sand at this point, which was very fine, a quicksand which when wet did not give sufficient foundation to withstand the pressure. All of the evidence seems to have been that the concrete was properly placed at the time, although no part of it or the pipe in question could be found.

The gates were closed from the main line near the accident in 25 minutes and water conditions throughout the city were normal within a few minutes of this time.

The large opening in the street, however, resulted in serious damage to the gas pipe where a 12-in. main was broken and all of the gas mains in the lower section of the city were flooded with water, resulting in an interruption of the gas service for about 36 hours. One circuit of the Electric Light Co. was also interrupted for about an hour.

The soil at this point is, when dry, of a fine, firm sand, but when moistened has the consistency of quicksand.

At the new Provin Mountain Reservoir, which stores filtered water to a depth of 26 ft., the water during the 25 minutes was lowered 2 ft. and the Venturi meter charts indicated that 1,300,000 gals. in excess of the usual use was wasted.

The opening in the street was about 60 × 30 ft. in plan and of a depth of from 10 to 12 ft. The damage to the streets leading to the river, and to the cellars, sidewalks and yards in the vicinity was also troublesome.

Notes from Engineering Schools.

RENSSELAER POLYTECHNIC INSTITUTE.—Negotiations have been completed for beginning the construction of the new building on the campus at Troy, N. Y., to be known as the Pittsburg Building in recognition of contributions received toward its construction from Pittsburg alumni. The new building will take the place of the old administration building, which is about half a mile from the campus, and has become entirely inadequate. It is planned to complete it within a year.

A SWITCH ENGINE FELL 150 FT. March 20 from a siding on the Crow's Nest Pass branch of the Canadian Pacific Ry., near Nelson, B. C. Three men in the cab were killed. The locomotive left the rails while shunting a string of freight cars, overran the edge of the embankment and dropped into the Goat River.

NEGLECTED TRAIN ORDERS caused a butting collision March 22 between two freight trains on the Louisville & Nashville R. R. at White Station, near Berea, Ky. Three trainmen were killed. An operator at Berea failed to deliver an order to sidetrack one of the trains at that place. The omission was discovered before the trains met and a wrecking crew was started for the scene before the accident occurred.

AN EXPLOSION IN A COAL MINE at Cambria, Okla., March 26, killed four miners.

A PREMATURE BLAST at Revelstoke, B. C., March 21, killed three workmen employed by the contractors for the dam under construction for that city's new hydro-electric plant.

THE PREMATURE DISCHARGE OF A 3-IN. GUN on the U. S. cruiser "Charleston," March 27, during practice at sea off Olongapo, P. I., caused the death of eight sailors. The gun was discharged accidentally in some unexplained way before the breech block had been locked shut. The block was blown from its hinges and into a group of sailors.

A GAS EXPLOSION DERAILED A LOCOMOTIVE on the Western Maryland R. R., March 21. A gas main had burst under the bed of the Potomac River near the Western Maryland roundhouse, and escaping gas was ignited a few minutes later by a passing yard engine. The explosion overturned the engine and the fireman was seriously burned.

FOREST FIRES IN SOUTHERN MISSOURI last week burned over thousands of acres of timber land and occasioned considerable additional loss to farmers through the destruction of buildings and fences.

A THREATENING FIRE ON THE MANHATTAN Bridge on March 25 damaged the structure to the extent of at least $25,000. The roadway of the bridge is still not quite completed; in the work of laying asphalt sidewalk surfacing in the Brooklyn land span, several kettles of hot asphalt were being used, when the contents of one of these caught fire from an ember blown by the strong wind. Creosoted track ties and guard-rails furnished fuel to the flames, and as the fire department found it slow work to get streams up to the fire, the heat became high enough to affect some of the floor steelwork. The main cables, trusses and suspenders were not affected, and the damage was confined to the stringers of one track on the lower deck, and possibly the floor-beams. A length of about 400 ft. of bridge was affected.

A COACHHOUSE USED AS A BALLROOM BURNED in the village of Oekoerit, Hungary, March 27, while a public ball was in progress, and 300 persons were killed. The coachhouse of the village hotel had been fitted up as a ballroom. A combination of dry branches and Chinese lanterns in the decorations started the fire. A burning branch fell to the floor and ignited the clothing of some of the dancers. This started a panic and stampede for the solitary exit, which had been nailed up, according to press reports, earlier in the evening to prevent the overcrowding of the ballroom.

FIRE DESTROYED THE TOWN OF MOUNT HOPE, W. VA., 45 miles southwest of Charleston, March 24. About 400 buildings were burned and only four were left standing.

The tannery of the Hambleton Leather Co., at Hambleton, W. Va., was destroyed by fire March 24. The loss is estimated at $1,000,000. The tannery employed 300 men.

At Hamilton, Mass., March 25, 21 buildings were destroyed by fire. Most of the buildings burned were dwelling houses and barns. The local fire department was unable to cope with the flames until assistance came from neighboring towns.

At Welch, La., March 27, 30 buildings in the business section of the town were burned, including the post-office building. The town had no water-works.

A FIRE IN A FURNITURE STORE BUILDING in Chicago, Ill., March 25, caused the death of twelve employees, eight of whom were girls. The building was occupied by the L. Fish Furniture Co. and is situated at 1906-8 Wabash Ave. The fire started on the fourth floor in the repairing and finishing room, in which benzine was stored, and burned out the floors and partitions of this floor and the two above it. On the top floor, the sixth, were the furniture company's offices, and all but one of those killed were on the sixth floor. There were two elevators (freight and passenger), one narrow stairway and an old-style ladder fire-escape communicating with this floor. Only two of the persons who were on the sixth floor when the fire started escaped alive. One slid down the cable of the freight elevator and the other descended the fire escape at the rear of the building to the fourth floor, and was obliged to drop the rest of the way to the ground. This man was the only one to attempt to use the fire escape, which was quickly enveloped in flames from the floor where the fire started. A number of men from the fifth and fourth floors escaped by jumping from fourth-floor windows to the roof of an adjacent three-story building.

The fire was started through the agency of an office boy who had been sent to the finishing room to fill three cigar lighters with benzine. This he was doing, from a five-gallon can, when the benzine was ignited, dropped the can and ran, escaping serious injury. The flames spread quickly over the entire end of the fourth floor and there seems to have been no attempt by any one in the building to extinguish them.

Seven months ago, a recommendation that this building be provided with additional fire escapes was submitted by the fire department to the Chicago Commissioner of Buildings, Mr. Murdock Campbell. In the report accompanying this recommendation, the building was described as being heavily loaded with furniture of inflammable material. No action on this recommendation had been taken by the building department.

SHIPWRECKS IN THE FALL AND WINTER SEASON of 1909-10 off the New England and British North American coasts involved 83 vessels and caused the loss of 225 lives. All but 13 of the ships lost were sailing vessels. Among the 62 schooners included in the list were four four-masted schooners, two five-masters and one six-master. The six-masted schooner "Mertie B. Crowley" of Boston, which registered 2,410 tons net, was the largest vessel lost. The unusually heavy loss this season was due in part to the exceptionally severe storm of Dec. 2.

THE POSITION OF ACCIDENT INSPECTOR has been incorporated into the reorganized claim department of the Chicago & Northwestern Ry. Mr. R. C. Richards, claim agent for many years, has been appointed to the new position. The duties of the accident inspector are the observation of causes of accidents with regard to the methods of handling trains and freight consignments and the elimination of these causes as far as possible by education, enforcement of discipline and encouragement of cooperation between railway officials and employees.

THE WILLOW CREEK DAM OF THE SUN RIVER project of the U. S. Reclamation Service is about to be put under construction. The dam will be 110 ft. high, 1,045 ft. long on the crest and will contain 437,000 cu. yds. of earth. The reservoir formed by the dam will have a capacity of 84,000 acre-ft. A railroad line has been projected for construction up the Sun River Valley to Augusta from Great Falls, Mont.

THE ATBARA BRIDGE IS TO HAVE NEW TRUSSES in order to provide for the increased live-loads of the present rolling stock on the Sudan Government Railways as compared with the 2,000 lbs. per ft. specified when the original contract was let, in 1899. The Sudan Government has just awarded a contract for the renewal to the Cleveland Bridge & Engineering Co., Ltd., of Darlington, England, according to a note in "The Engineer" of London, March 11, 1910. The original structure was built by the Pencoyd Iron Works, of Philadelphia, Pa. It consists of seven 147-ft. pin-connected spans, single-track, resting on steel cylinder piers. The floorbeams and stringers are still ample for the service, but the main trusses are overstressed. New trusses are, therefore, to be provided, using the existing shoes and receiving the present floorbeams. The present panel lengths will be retained, but the height of truss is increased to 26 ft. To secure greater lateral stiffness the web-members are to be made 12 ins. wide, which is a trifle wider than the present ones. This extra width, and the thickness of a gusset-plate on either side, will make the spacing of trusses some 2½ ins. greater than the present spacing. The shoes will not be shifted, however, but the upper shoe member is to be rebuilt to permit of the offset. To maintain a minimum clearance of 15 ft. width (as against c. to c. spacing of 16 ft. 4½ ins.) the end posts can not be as wide as the top chord, but will be similar to the vertical web-members. The trusses are to be designed for carrying future sidewalk brackets. Holes for turned bolts (in tension) will be provided in the posts for this purpose. The reconstruction of the bridge is to be completed by June, 1911. The contract includes a temporary bypass bridge for carrying the railway traffic during the reconstruction.

A 32-MILE TUNNEL underneath the Cascade Range in Washington, to make connection between the eastern side of the state and the coast region, is proposed by Gen. H. M. Chittenden, Corps of Engineers, U. S. A., retired, of Seattle, Wash. Gen. Chittenden printed an anonymous pamphlet some months ago describing his scheme, the pamphlet taking the form of a paper read before an engineering society in Seattle at a meeting in 1925, the author of the paper being one "Itoth Phucher" who is supposed to be the chief engineer of the hypothetical tunnel. The cost of the tunnel is estimated at a million dollars a mile, and the cost of 327 miles of connecting railway lines at $90,000 per mile brings the total estimate of cost to 63½ million dollars. Equipping the lines for electric operation is assumed to raise the total to 75 million dollars. In addition to accommodating the railway traffic through the tunnel, special cars would be used for transporting vehicles through it. The tunnel would be driven from the two end portals and two intermediate shafts. The longest section between headings would be 13 miles, which, it is estimated, could be driven in six years. Since the recent disastrous ac-

nts on the Great Northern Railroad across the mountains, due to snow avalanches, Gen. Chittenden has made s scheme public, an article on it appearing in the attle "Post-Intelligencer" of March 5.

THE WHITE MOUNTAIN AND APPALACHIAN rest reserve bill was favorably reported to the House Representatives on March 23. The committee did ded 10 to 7 on the question whether the bill should be vored. The bill appropriates $9,000,000 to be spent uring five years to acquire timber lands on the watersheds of navigable streams flowing from the Southern ppalachian and White Mountain regions.

A BILL TO RAISE THE WRECK OF THE BATTLEship "Maine," sunk in the harbor of Havana, was passed y the House of Representatives on March 23.

A SPEED OF 131.7 MILES PER HOUR over a measured mile was attained by an automobile of 200 HP., on he beach at Daytona, Fla., on March 16. This exceeds he speed record attained by any other method of transportation.

STREET-CAR FENDERS are now required on all lines n San Francisco. An ordinance requiring their use after April 30 was passed by the Board of Supervisors on March 14.

GOVERNMENT GRAIN ELEVATORS in Manitoba are provided for in a bill recently introduced in the Manitoba Legislature. The bill is a response to the demand of the Grain Growers' Association.

A HIGHWAY TUNNEL AT NEWHALL PASS, CAL., described in the Los Angeles "Express," will be 430 ft. in length. The present highway over the pass is stated to have a grade of 23%. The new road through the tunnel cuts down this grade to 6%.

THE RALDIRIS BILL to restrict the practice of Civil Engineering in New York to graduates of engineering schools has been shelved for the present. At a public hearing on the bill on March 24 by the Assembly Committee on Education, a score of prominent engineers were present to oppose the bill, including representatives of the American Society of Civil Engineers, the Engineering Society of Eastern New York and the Albany Society of Engineers. After an informal discussion, Mr. Raldiris, the introducer of the measure, stated that in view of the many objections which had been raised, he would not push the bill in its present form, but that he would arrange for a conference with representatives of the various engineering societies with a view to formulating a measure in which the objectionable features of the present bill would be eliminated, and that a bill, to be agreed upon, would be substituted for his present measure.

ELECTRIC TRACTION on the Harlem branch of the New York Central & Hudson River R. R. was formally opened to North White Plains on March 14, though trial trains had been in operation for several days previous and electric trains had been running as far as Wakefield (12 miles) for several months. This completes the length of proposed change on this suburban branch, a distance of about 22 miles from Grand Central Station, New York City.

THE BUREAU OF MUNICIPAL RESEARCH IN NEW York City has been requested by Mr. John F. Fitzgerald, the new Mayor of Boston, Mass., to confer with that official regarding "the constructive measures and methods that are being installed and tested under the advice of your Bureau in other cities."

TELEPHONE AND TELEGRAPH COMPANIES IN New York State will be placed under the jurisdiction of the Public Service Commission of the Second District, if the Legislature carries into effect the unanimous recommendation of the special legislative committee appointed last year to investigate a report on this subject. The committee was of the opinion that since the telephone and telegraph companies extend over the whole state, the Public Utilities Commission of either one or the other of the two districts should be put in charge instead of having divided responsibility.

A FILTERED WATER-SUPPLY FROM THE MISSISsippi River has been recommended for the city of Minneapolis by Mr. Rudolph Hering, M. Am. Soc. C. E., who was called in some time ago to report on the relative merits of such a supply and of a new supply from the Mille Lacs Lake. Mr. Hering found that the quantity of water available from the lake source had been greatly overestimated. Due to the heavy evaporation and the relatively large area of the lake, the annual evaporation is 6½ ins. greater than the estimated rainfall of 27½ ins. Accordingly, a daily supply of only 10,000,000 gals. could be depended upon from this source, and so small a supply would not justify the construction of necessary works, which would involve a 64-in. pipe line 80 miles in length. Moreover, filtration of the Mille Lacs supply would be necessary to remove fine suspended matter and opalescence after storms, besides which hypochlorite treatment might be required. Mechanical filtration is advised for the Mississippi River water, apparently supplemented by hypochlorite treatment. It is also suggested that a softening plant might be desirable and that this could be added at any time. The estimated cost of the Mille Lacs project was $6,822,140 of capital outlay, as compared with only $522,500 for the Mississippi River project, with filtration. The comparative yearly charges, including allowances for capital, are $432,126 for the Mille Lacs project and $163,350 for the Mississippi River project, thus making a difference of $268,766 in favor of the latter.

THE RECENT TYPHOID FEVER OUTBREAK AT Montreal has been put fairly and squarely upon the water-supply by an investigating committee working under the auspices of the Emergency Typhoid Hospital. The committee appears to have been selected on request by the medical faculties of McGill and Lava Universities, Montreal, and was made up as follows:

J. G. Adami, Professor of Bacteriology and Pathology; G. E. Armstrong, Professor of Clinical Surgery; R. F. Ruttan, Professor of Chemistry; T. A. Starkey, Professor of Hygiene, representing McGill University, and A. Bernier, Professor of Bacteriology; J. J. Guerin, Professor of Clinical Medicine; Henri Hervieux, Professor of Practice of Medicine; E. P. Lachapelle, Dean and Professor of Hygiene, representing the University of Laval.

The committee declared that it found

the evidence overwhelmingly that the chief cause of the disease is the water-supply of the city and its suburbs.

We quote further as follows:

The exhaustive investigations into the condition of the water-supply of the city indicate that while the water may give favorable analyses for some months of the year, none of them are at all times safe. At certain times of the year the waters are actually bad and capable of giving rise to water-borne diseases.

We do not consider that any further analyses or examination of the waters are necessary to establish the above conclusions.

It follows then that the future water-supply of the city, whether it consists of unmixed St. Lawrence from south of Nun's Island or not, must be purified by a thorough system of filtration before distribution to the public.

To ensure the proper working of the plant, constant supervision of this filtered water by an expert water analyst, devoting his entire time to the work, is an essential part of such a system.

SIX 20,000-KW. GENERATORS will be installed in a new power station which the Commonwealth Edison Co., of Chicago, will build on the North Branch of the Chicago River near Roscoe St. and the Extension of California Ave. This site is some six miles northwest of the business center. Work will be begun on this station in the next few weeks and the initial installation will be of two of the 20,000-KW., steam-turbine units with a later increase to six. It is also a part of the announced plans to build a second and duplicate station, on space retained at this site, at some later time. Each station will cost about $10,000,000 and will be built on the unit plan, each unit consisting of one turbo-generator, ten boilers, with a single stack and all necessary auxiliaries. The turbines will be vertical Curtis turbines driving 2,300-volt, 25-cycle generators at 750 r. p. m. These are the largest generating units yet proposed. They will be 15 ft. in diameter and 36 ft. high, but projecting above the engine room floor only 28 ft. The condenser for each turbine will be under it and will be of greater diameter. Each unit will require 260,000 lbs. of steam per hour at 250 lbs. per sq. in., while fully loaded.

The buildings will be of red brick like the other Commonwealth stations, and especial attention will be given to the architectural treatment of the exteriors, it is said. It is proposed to line the boiler and engine rooms with white enameled brick. When the two stations are completed, a small administration building will be erected between them. This will have a dining-room, kitchen and sleeping rooms in addition to the usual offices.

In the rear of each station space is provided for the open storage of 150,000 tons of coal—a 60-day's supply. This coal will be handled by a gantry crane. Tracks will be extended from the Chicago and Northwestern Ry. which is close by the site. A concrete tunnel 700 ft. long will run from each station to the river for the condensing-water supply.

FIXED-RATE LIGHTING for small residences is being undertaken by a company, "The Fixed Price Light Co., Ltd.," operating in England and formed under the auspices of Siemens Brothers Works. Owners or tenants of small places may have their buildings wired and energy supplied by this concern under an arrangement by which they are merely charged a fixed rental per lamp. The rentals are to include, then, the cost of wiring, fixtures and of the first lamps installed. Lamps subsequently used must be paid for by the customer. This last feature is included to prevent excessive burning of the lamps which, of course, is shown by their reduced life. The charges have been fixed at 5 cts. per lamp per week, April to October and 7 cts. per week during the darker months. One tantalum lamp, with shade, will be provided in each room desired and it is specified that the illuminating power of the units installed shall enable sewing on black fabrics to be comfortably done at common table levels. A feature of the system is the use of the Siemens "concentric" conductors, with which an outer sheath serves as return wire. It is stated that the fixed-rate system relies on this feature to reduce installation costs to the necessary level for success of the undertaking.

THE DISPOSITION OF PREMIUMS ON MUNICIPAL bonds in Massachusetts will be established beyond question if a bill introduced in the Legislature on the petition of Mr. Charles W. Sherman, M. Am. Soc. C. E., becomes a law. The bill states that the premiums are municipal bonds of definite character. After deducting the "cost of preparing, issuing and marketing" said bonds shall be applied to the sinking fund for the redemption of the bonds, and in case there is no sinking fund, but instead the bonds mature yearly, then the premium shall be applied to the payment of the first bond to become due.

THE PENNSYLVANIA RAILROAD SYSTEM, according to the Annual Record of the transportation lines included in it, just issued, now comprises 24,098 miles of track and 11,234 miles of railway line. There are 3,348 miles of second track, 1,330 miles of third and fourth tracks and 8,185 miles of company sidings. In total length of tracks the Pennsylvania system is by far the largest railway system in the world. The Pennsylvania lines extend through 14 different states. The mileage located east of Pittsburg and Erie is 6,294 miles and this exceeds considerably the mileage of the system west of these points, which is 4,940 miles. The increase in trackage of the system during 1909 was only 120.53 miles.

THE COST OF FOREST-FIRE FIGHTING may be estimated from the Forest Service's figures concerning the national forest reserves, for 1908 and 1909. The cost of fighting was $73,000 in 1908, excluding the pay of forest-rangers and guards on fire patrol duty. The fires which occurred in 1908 numbered 2,728, in 1909 no less than 3,138. These included some very extensive fires, so that while over 80% of them did not exceed five acres in area, the average of all is 150 acres per fire in 1908, 120 acres per fire in 1909. The large proportion of cases in which the fire was held down to a very small area is taken as evidence of the value of the fire-patrol and fire fighting service. The total damage done was about $450,000 in 1908, $300,000 in 1909, in standing timber destroyed; and $700,000 in 1908, $160,000 in 1909, in damage done to reproduction and forage.

STATIONARY ENGINE RUNNERS IN MANITOBA have been seeking legislation from the provincial legislature requiring every engineer operating an engine to have a license, such license to be granted by an incorporated association of the engineers, which should also have power to conduct examinations to determine the fitness of an applicant to receive a license.

CAR LIGHTING BY INCANDESCENT GAS LAMPS, or gas-mantle lamps, is used on the Eastern Railway, of France, and a system of electric-spark apparatus is being introduced for lighting the lamps simultaneously. This is a substitute for the old and awkward practice of having each lamp lighted by hand by a man running along the roof of the train or entering each compartment of every car when standing at a station. This practice was not only awkward, but it involved considerable delay, the time of lighting was often inconvenient, and the slamming of the lamp covers caused shocks which reduced the life of the mantles. With the new system, all the lamps of a car (or a train if the cars are electrically connected) are lighted simultaneously by a trainman operating the lever of an electrical apparatus fitted to the car. In one type of apparatus this is done automatically by the opening of the gas valve. The lamps can be lighted and extinguished at any time, to suit the varying conditions of season and weather, and also when the train is approaching a tunnel. Wires connecting with a battery and simple electrical apparatus are led along the roof of the car, and a branch wire is led to each lamp, which has a sparking device placed just above the gas mantle.

The system is considered specially adapted for cars run in interchange service on large routes, but it has been found very advantageous also for the trains of double-deck enclosed cars operated in the Paris suburban service. Among the favorable results are a reduction in the consumption of gas, an increase in the life of the mantles, a reduction in cost of lighting and maintenance, and the elimination of the risks run by the men working on the roofs of the cars. The system is described in some detail in the February number of the "Revue Generale des Chemins de Fer" in a paper by Mr. E. Biard, Chief Engineer of Rolling Stock, and Mr. G. Mauclere, Inspector of Rolling Stock, of the Eastern Railway.